

# **SIMPLIFIED DESIGN OF CONCRETE FLOOR SYSTEMS**

*With Design Tables*



**PORTLAND CEMENT ASSOCIATION**







# **SIMPLIFIED DESIGN OF CONCRETE FLOOR SYSTEMS**

*With Design Tables*

•

*Published by*

**PORTLAND CEMENT ASSOCIATION**

**33 WEST GRAND AVENUE • CHICAGO, ILLINOIS**



# CONTENTS

•

<i>Section</i>	<i>Subject</i>	<i>Page</i>
I	Concrete Floor Systems and their Characteristics	5
II	Elements of Design	13
III	Structural Design Tables	39
IV	Special Details of Construction	82

<i>Problem</i>	<i>Subject</i>	<i>Page</i>
1	Design of Solid One-Way Slab and Supporting Beams	20
2	Alternate Design of Solid One-Way Slab	26
3	Design of One-Way Clay Tile Filler Ribbed Slab	28
4	Alternate Design of One-Way Clay Tile Filler Ribbed Slab	30
5	Design of One-Way Metal Pan Filler Ribbed Slab	33
6	Design of Solid Two-Way Slab and Supporting Beams	36

# ILLUSTRATIONS

•

<i>Figure No.</i>	<i>Subject</i>	<i>Page</i>
1	Bending moment and shear coefficients for concentrated loads	15
2	Sketch of rectangular beam, showing distribution of stress	17
3	Sketch of rectangular beam reinforced for compression, showing distribution of stress	18
4	Sketch of T-beam, showing distribution of stress	19
5	Layout of panel, to accompany Problem 1.	20
6	Cross section of slab, showing bar bends, to accompany Problem 1	21
7	Layout of panel, to accompany Problem 2.	25
8	Layout of panel, to accompany Problem 3.	26
9	Layout of panel, to accompany Problem 4.	31
10	Layout of panel, to accompany Problem 5.	34
11	Layout of panel, to accompany Problem 6.	35
12	Bar bending diagram for solid slabs	41
13	Bar bending diagram for joist slabs	43
14	Bar bending diagram for T-beams	44
15	Framing plan and bar schedule for illustrative example for two-way reinforced slab	47
16	Factors for distribution of slab load to beams for two-way reinforced slab	49
17	Diagram for stirrup spacing	69
18	Arrangement of reinforcing around small slab openings	82
19	Section of stairs and platforms, to show arrangement of reinforcing	83
20	Right and wrong methods of anchoring reinforcing bars	84



# TABLES

<i>Table No.</i>	<i>Subject</i>	<i>Page</i>
1	Bending Moments in Slabs, Beams and Girders for Uniform Loads . . . . .	14
2	Summary of Quantities and Panel Loads. . . . .	39
3	Values of Design Constants for Various Combinations of Steel and Concrete Stresses . . . . .	39
4	Rectangular Beams with Compression Reinforcement Design Values of $p$ and $K$ . . . . .	40
5	Sectional Area, in Sq. In., of Various Numbers of Bars . . . . .	40
6	Area of Steel, in Sq. In. per Linear Foot, of Slab for Bars Spaced At Various Intervals . . . . .	41
7	One-Way Solid Concrete Slabs . . . . .	50
8	Concrete or Clay Tile and Concrete Joists . . . . .	52
9	20-in. Metal Pans and Concrete Joists (Straight Pans) . . . . .	54
10	20-in. Metal Pans and Concrete Joists (Tapered Pans) . . . . .	56
11	30-in. Metal Pans and Concrete Joists (Straight Pans) . . . . .	58
12	30-in. Metal Pans and Concrete Joists (Tapered Pans) . . . . .	60
13	Concrete T-Beams (Simple Spans) . . . . .	62
14	Concrete T-Beams (Two Spans). . . . .	66
15	Concrete T-Beams (End Spans). . . . .	67
16	Concrete T-Beams (Interior Spans) . . . . .	68
17	Two-Way Solid Concrete Slabs (Square Panels). . . . .	70
18	Two-Way Solid Concrete Slabs (Ratio of Spans 1.2 to 1.0). . . . .	72
19	Two-Way Solid Concrete Slabs (Ratio of Spans 1.4 to 1.0). . . . .	74
20	Two-Way Solid Concrete Slabs (Ratio of Spans 1.6 to 1.0). . . . .	76
21	Two-Way Solid Concrete Slabs (Ratio of Spans 1.8 to 1.0). . . . .	78
22	Two-Way Solid Concrete Slabs (Ratio of Spans 2.0 to 1.0). . . . .	80
23	Concrete Stair Slabs . . . . .	83
24	Recommended Live Loads . . . . .	85
25	Weights of Building Materials . . . . .	86

1D 88-B12240 Corrm 10



*To those who design*  
**ECONOMIC FLOORS:**

**T**HE availability and economy of materials used in reinforced concrete make this form of construction ideal for floors for all uses. The very nature of concrete permits a flexibility in the size and arrangement of floor members. It materially lowers insurance rates.

Concrete floors are rigid and have great load carrying capacity when correctly designed. With the proper finish these floors are easily kept clean, and wear is negligible. Their long, useful life with minimum maintenance cost produces an ideal economic combination.

This booklet enumerates and discusses various concrete floor systems, shows their respective advantages and relative economics, and illustrates the method of design by typical examples. Different floor constructions suitable for a set of specific requirements are analyzed to illustrate the procedure in determining the most economical type.

Safe load tables of numerous types of slabs and beams are given to assist the designer by simplifying the calculations involved. These tables are based on requirements in the building code of the American Concrete Institute, as this code has come into quite general use. Differences between this code and local building codes, if any, must necessarily be taken into account by the designer.

*Portland Cement Association*



# Simplified Design of Concrete Floor Systems

## SECTION I—CONCRETE FLOOR SYSTEMS AND THEIR CHARACTERISTICS

**T**HE choice of a type of concrete floor for any specific use involves the careful consideration of points characteristic of each system. Intelligent comparisons may then be made and selection of a suitable, economical floor becomes a relatively simple matter. The elimination of types not inherently adapted to the proposed use reduces the number for which comparative cost studies should be made.

The most important points to be considered are:

### *I. Use of Floor*

1. Loading.
  - a. Light or heavy.
  - b. Uniform or concentrated.
2. Span.
  - a. Long or short.
  - b. Regularity of support arrangement.
3. Freedom from obstruction.
  - a. Columns.
  - b. Beams or girders.
  - c. Incorporation of pipes, conduits or vent shafts.
4. Rigidity and resistance to:
  - a. Machinery shocks.
  - b. Outside vibrations.
  - c. Wind stresses or earthquakes.
5. Adaptability to alteration.
  - a. Main members.
  - b. Secondary members.



6. Conformity to code.
  - a. City or other local codes.
  - b. Fire or other insurance codes.
7. Insulation.
  - a. Sound.
  - b. Heat or cold.
8. Acoustical properties.

## ***II. Cost of Floor***

1. First cost.
2. Maintenance cost.
3. Indirect cost. (Affecting cost of remainder of building.)
  - a. Ability to "tie in" to structure.
  - b. Difference in weight and story height as affecting size and cost of supporting columns and foundations.
4. Royalty costs of patented types.
5. Salvage of formwork.

## ***III. Appearance of Floor***

1. Floor finish.
  - a. Integral concrete.
  - b. Any finish applied later.
2. Ceiling finish.
  - a. Exposed and untreated.
  - b. Plaster.
  - c. Paint.

## ***IV. Local Building Conditions***

1. Material.
  - a. Supply, including freight rate.
  - b. Handling on the job.
  - c. Storage space.
2. Labor.
  - a. Type available.
  - b. Possibility of importation.
  - c. Labor restrictions.
3. Special conditions of bidding by contractors.
4. Climatic conditions.



### ***V. Time Requirements***

1. Speed of construction as affecting revenue of the building.
2. Delays caused by purchase of special materials not available locally.

### ***VI. Supervision***

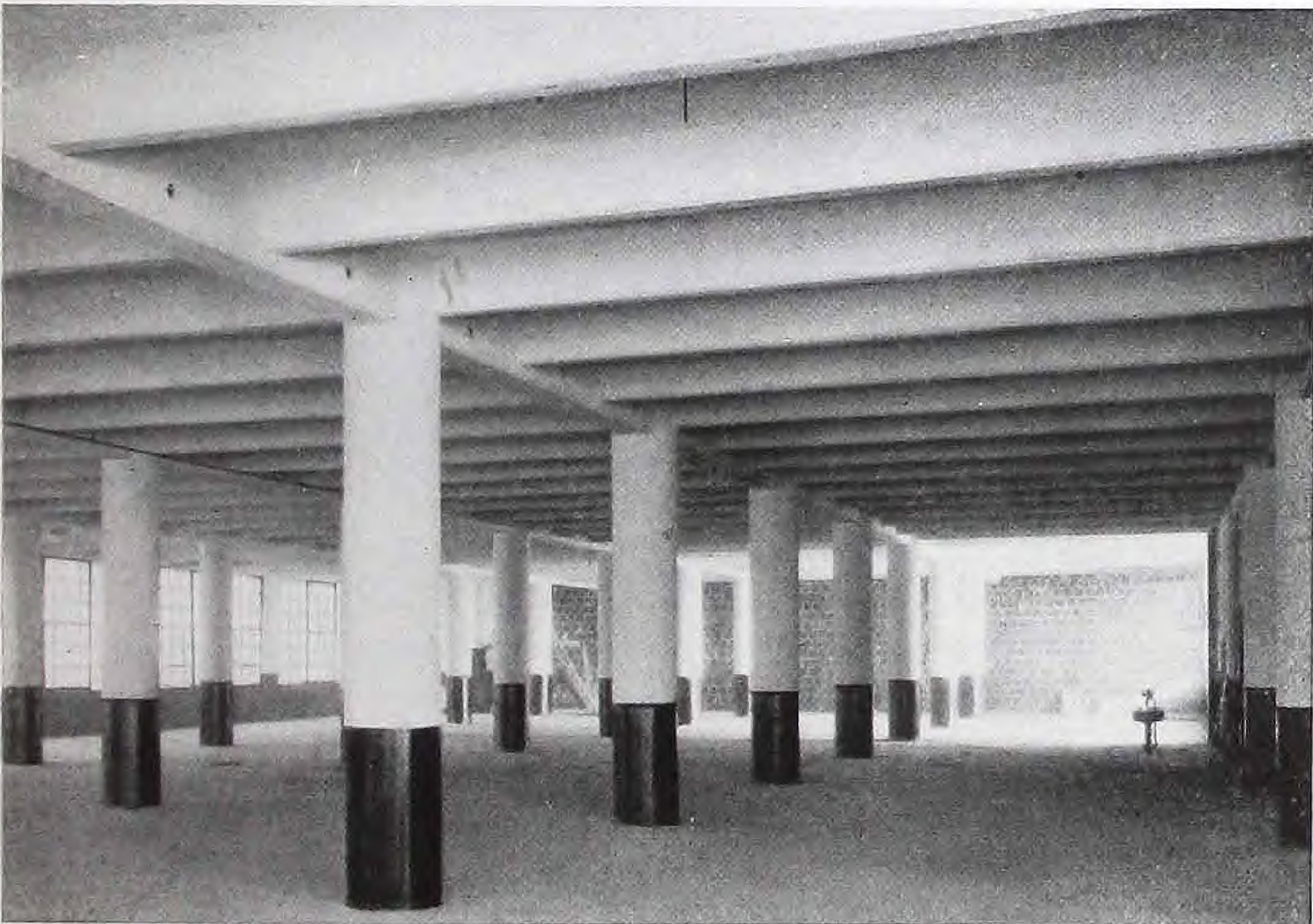
1. Normal.
2. Extra rigid, due to unusual or new system involved.
3. Ease of repairing the result of slight errors or omissions, without reducing factor of safety.

Several of the more commonly used systems are enumerated in the pages to follow, with descriptions, general characteristics and advantages for special purposes or unusual conditions. To facilitate discussion and without intention of fixing definite limits, live loading for floors, as referred to in this review, will be divided into three groups and defined as light (from 40 or 50 to 100 p.s.f.\*), intermediate (100 to 175 p.s.f.) and heavy (any live load in excess of 175 p.s.f.).

### ***One-Way Solid Slab***

Concrete slabs of uniform depth, with no filler material, and with the main reinforcement in one direction only, are called one-way solid slabs. The term "flat slab" is often erroneously applied to such slabs.

\*p.s.f. is an abbreviation of "pounds per square foot."



**One-way solid slab construction produces rigid, economical floors suitable for industrial plants and buildings of other occupancy.**



Constructed in this manner such slabs constituted the earliest application of concrete to a floor system. They have many good points which make them *suitable and economical when used for supporting intermediate and heavy loads on comparatively short spans*, say, from 6 to 12 ft. long. For light loads these spans may be increased to 14 or 15 ft.

Construction progresses rapidly, as all of the materials used are easily procured, and only ordinary labor and normal supervision are necessary.

Solid slabs produce a stiff and rigid floor capable of withstanding vibration and shock from machinery or earthquakes, and are adapted to use where heavy concentrated loads must be supported.

They provide an excellent base for concrete floor finishes, are easily kept clean and are allowed a low insurance rate because of their high fire resistive qualities.

Limitations to the use of one-way solid slabs for long spans are due to their comparatively large dead weight. Also, the introduction of more beams than used in other systems may not meet the architectural requirements for the floor layout below. The slab weight may be considerably reduced by the use of lightweight aggregate. This reduction may effect a pronounced saving in the indirect cost of the structure (i.e., in reducing the sizes of columns and footings). Lightweight aggregate also improves heat insulation.

If good tight forms are constructed of pressed wood fiber boards or similar material, plaster may often be omitted, as an attractive ceiling surface results from merely painting or staining the exposed concrete.

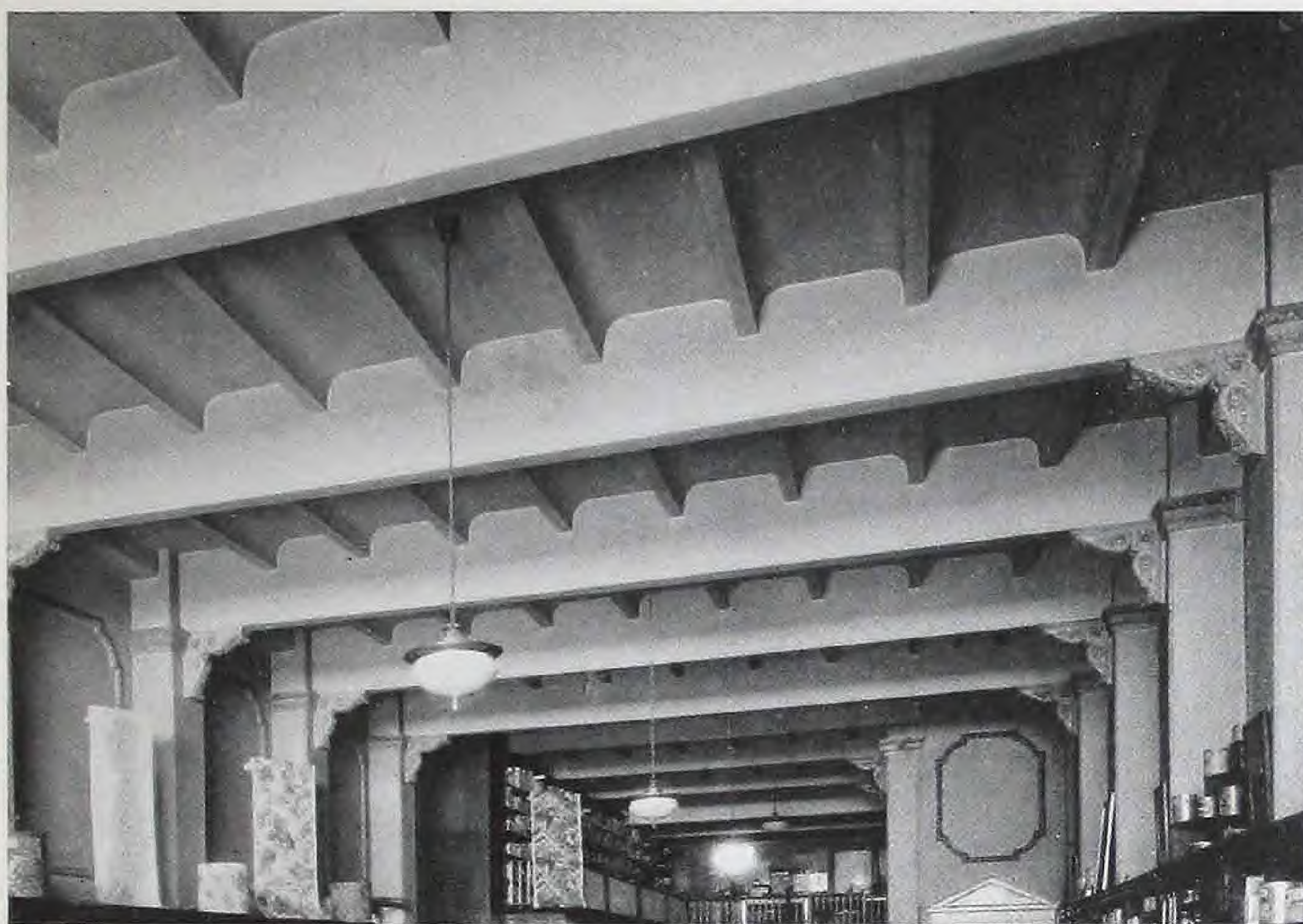
Ordinary electric conduits and pipes of about the same size can generally be run in the slab, but mechanical equipment of larger size may require space provided for by either a suspended ceiling or an extra fill between the slab and the finished floor.

### ***One-Way Ribbed Slabs with Block Filler***

Hollow filler blocks of lightweight concrete or clay tile, when laid in rows in the bottom of concrete slabs, constitute one form of ribbed slab. The dead load is considerably reduced as compared with a solid slab of equal load carrying capacity although the total depth of slab is increased. The width of the concrete joists, separating the rows of filler and encasing the reinforcement, may be made any desired width to meet the strength requirements. It is customary to include a solid concrete top of two inches or more in depth over the blocks. This serves the double purpose of providing a space for concealing small pipes and conduits and together with the joists gives a T-section, thereby adding considerable strength to them. Introduction of the filler blocks improves sound and heat insulation of slab.

While plaster may be applied directly to the slab, it has been noted that at times a slight discoloration takes place when clay tile filler is used. This is due to the use of materials of different densities and absorption qualities next to one another and may be eliminated by placing tile soffit pieces in the bottoms of the joists to form an all-tile ceiling.





**Ribbed slabs constructed with removable metal pans make a lightweight floor of pleasing appearance.**

Ribbed slabs of this type are *well adapted to floors of medium span, with light and intermediate loadings*. They cannot carry heavy concentrated loads as well as the solid slabs, and become relatively less economical when the spans are increased.

To insure straight joists, care should be taken that the filler blocks are maintained accurately in line. Owing to the porous nature of the blocks, they should be thoroughly sprinkled to prevent absorption of the water in the concrete, particularly in warm weather.

### ***One-Way Ribbed Slabs with Metal Pans***

One of the lightest types of concrete floors results from the use of metal pan fillers between concrete joists. Numerous kinds of such pans are available. Some are fabricated of light gauge material and are intended to be left in place. They may be procured with metal lath already attached at the bottom, upon which plaster may be applied directly. Other pans are made of heavier metal and are removed when the forms are stripped, to be used again or returned to the owner, if leased. Both styles are furnished with either straight or tapered closed ends. Tapered pans are highly desirable on long spans, as with their use the width of the joists is increased at points where such width is needed, namely, near the supports where the shearing stresses are highest. The usual width of metal pans is 20 inches, although 10, 15 and 30-in. pans are generally available.





Ribbed slabs often have spreader joists at third points of long spans. They are economical for light loads.

*Metal pan ribbed floors reach their highest economy on long spans with light loads. While the weight is very light, the depth of the floor produces great rigidity. Formwork is economical, merely requiring a board under each rib or joist, with the space under the pans left open.*

These floors are not so well suited to support concentrated loads, as the topping between joists is comparatively thin ( $2\frac{1}{2}$  to 3 inches). Care must be taken to reinforce the topping across construction joints to prevent these joints from opening.

### ***Two-Way Solid Slab***

Solid concrete slabs reinforced in two directions, and carrying load both ways to marginal beams, offer a system that is quite economical for certain uses and has other decided advantages. Modern building codes present improved design methods which make this type of floor economical and *well suited to support intermediate and heavy loads on spans up to about 30 ft.* Best results are obtained when panels are nearly square, although panels having a ratio of long to short sides of 2 to 1 may be designed.

Marginal beams framing in two directions to columns, while presenting possible architectural difficulty as to floor layout below, produce effective horizontal bracing. Wind and earthquake forces as well as vibrating machinery loads are properly resisted, and the slab is well suited to carry heavy concentrated loads through so-called "plate action."



Other advantages of the two-way slab are its high fire-resistive qualities, simplicity of formwork and ease of erection. If lightweight aggregate is used, the dead load may be reduced about 30%. This reduction will be reflected in lowering the indirect costs throughout the building structure.

### ***Two-Way Ribbed Slabs with Block Filler***

The introduction of hollow filler blocks of lightweight concrete or clay tile, with concrete joists running in both directions, results in *a type of floor having nearly all the advantages of both the one-way ribbed slab and the two-way solid slab*. Dead load is considerably reduced, but the total floor



**One-way and two-way ribbed slabs with block fillers produce good surfaces for plastering. Note the soffit blocks beneath the concrete ribs.**

thickness is increased. Care should be taken with this type, if clay tile filler is used, that the concrete is of such consistency as not to flow into and fill up the cells in the tile.

Other remarks made for one-way ribbed slabs with block filler will in general apply to the two-way ribbed slabs.

### ***Girderless or "Flat Slab" Floors***

Solid concrete slabs reinforced in two or more directions, but without marginal beams of any kind, are called "flat slabs." They have all *the economy, high efficiency and rigidity of the two-way slabs* previously men-





Maximum story height without obstruction is obtained with flat slab floor construction.



Painted flat slab ceilings are suitable for department stores and industrial buildings and are economical for the heaviest loads.



tioned, *but in even higher degree*. The absence of beams offers the additional advantages of better lighting, ventilation and arrangement of mechanical equipment if used in factory types of buildings. It is customary to employ supporting columns with flared heads or capitals. Also in some cases, it will be found that economy results from the use of panels of additional thickness at each column, called "drop panels."

Flat slabs make rigid, *efficient floors* for such heavy and concentrated loads as are encountered in *warehouses, loft buildings and industrial buildings with heavy or vibrating machinery*. Panels of medium span and nearly square are preferable. The slab must be made continuous over two or more spans.

Being girderless, the flat slab requires the simplest formwork only and is given a high fire-resistive rating due to the absence of projecting corners or edges. Story heights may be considerably reduced on account of the greater available clear ceiling height.

## Summary

While the previously described floors by no means exhaust the available types from which to choose, it will be found that in the majority of cases, at least one of those mentioned will provide an adequate, suitable and economical system; one that will give many years of service at a negligible maintenance cost.

## SECTION II—ELEMENTS OF DESIGN

Prior to the actual design of a floor, the designer should be thoroughly familiar with the structural provisions of the building code he is to use. The safe load tables and other data given in this section and in Section III are based on provisions of the Tentative Building Regulations for Reinforced Concrete, American Concrete Institute,\* referred to hereafter as the A. C. I. code, and concrete having an ultimate compressive strength of 2,000 p.s.i.\*\* at an age of 28 days. This concrete has an allowable unit stress of 800 p.s.i. on the extreme fiber, with an increase to 900 p.s.i. on the extreme fiber adjacent to supports of continuous or fixed beams and slabs. The unit stress of the reinforcement is taken at 20,000 p.s.i. in tension, and  $n = 15$ . The shearing stress in the concrete is limited to 40 p.s.i. without web reinforcement and bond stress to 100 p.s.i. for deformed bars. Allow-

\*Report of American Concrete Institute Committee 501, Standard Building Code, adopted by the Institute at its 32nd annual convention, Feb. 1936, as a tentative standard.

\*\*p.s.i. is a generally accepted abbreviation of "pounds per square inch."



able tensile stress in the web reinforcement is taken at 20,000\* p.s.i. In all cases, the amount of protective covering of the reinforcing steel is shown in the tables. Bending moments and shears for continuous beams and slabs with uniform load are given in Table 1.

TABLE No. 1—BENDING MOMENTS IN SLABS, BEAMS AND GIRDERS FOR UNIFORM LOADS

Coefficients of  $wl^2$

Number and Length of Spans		End Spans		Intermediate Spans	
		+M at center	−M at 2nd support	+M at center	−M at support
Spans equal to or less than 10 ft.	Two spans . . . . .	1/10	1/10	.....	.....
	More than two spans	1/10	1/12	1/12	1/12
Spans greater than 10 ft.	Two spans . . . . .	1/10	1/8	.....	.....
	More than two spans	1/10	1/10	1/12	1/12

Note: Shear in end members at the 2nd support is  $1.20 \frac{wl}{2}$  and the shear at all other supports is  $\frac{wl}{2}$

For bending moments and shears in continuous girders with concentrated loads, the coefficients given in Figure 1 are used.

The coefficients in Table 1 and Figure 1 may be used for spans in which the longer of two adjacent spans does not exceed the shorter by more than 20 per cent, and in which the intensity of the live load is not more than three times the intensity of the dead load. In these cases,  $l$  equals the average of two adjacent clear span lengths for negative moment and the applicable clear span for positive moment.

Negative reinforcement to be provided at the outer end of all members built integrally with their supports shall be not less than  $0.005\ b'd$  for T-beams and  $0.005\ bd$  for rectangular beams and slabs.

A. C. I. Code recommendations with regard to “slabs supported on four sides” are the basis of values in the tables of two-way solid slabs. They take into account the condition of continuity of the slab, the ratio of the long side to the short side in rectangular panels, and the so-called “plate action” of the slab.

\*See footnote\*\* on page 22.



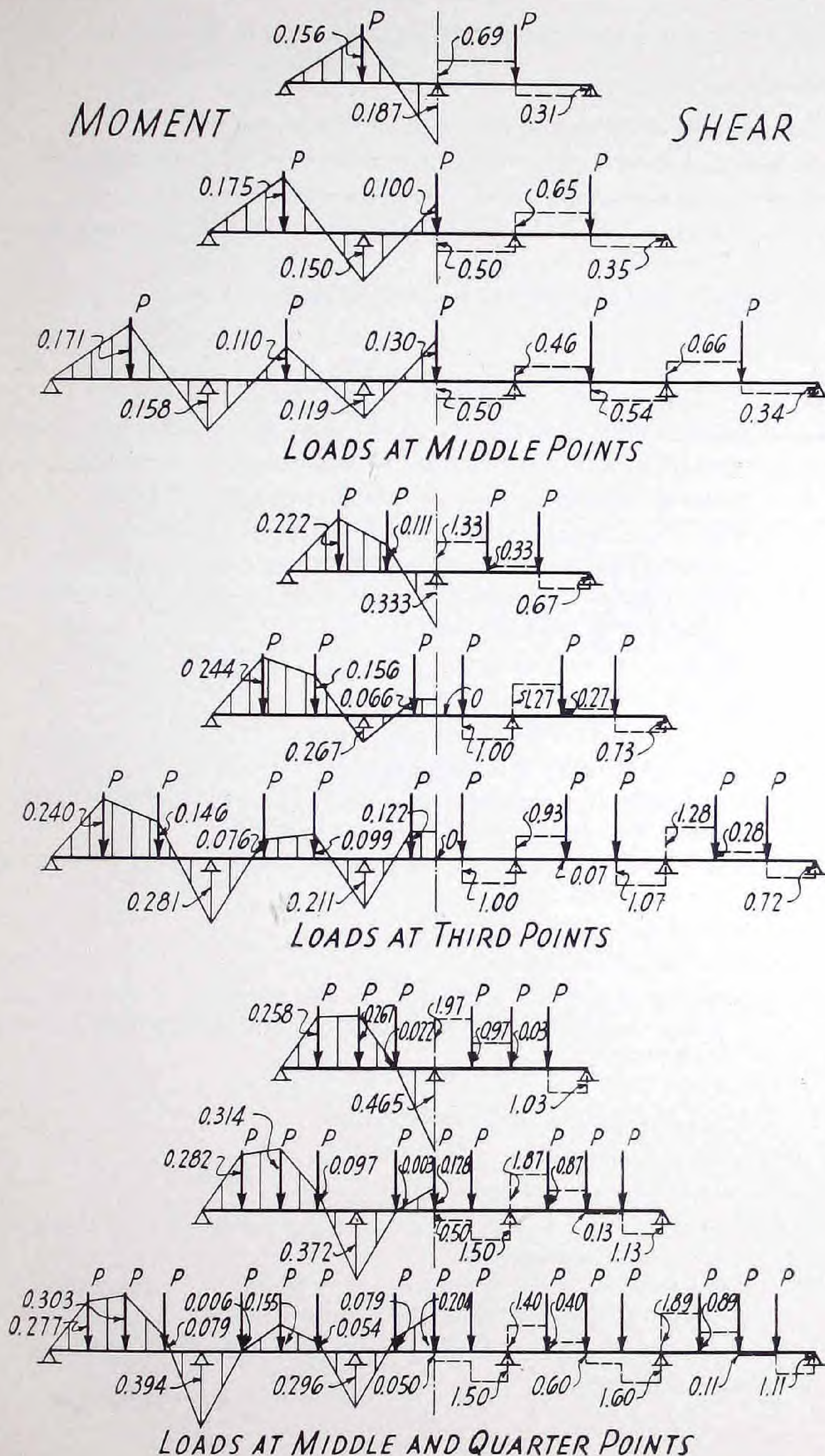


Fig. 1



## FORMULAS FOR REINFORCED CONCRETE DESIGN

### *Standard Notation*

$f_s$  = tensile unit stress in longitudinal reinforcement.

$f_c$  = compressive unit stress in extreme fiber of concrete in flexure.

$f_v$  = tensile unit stress in web reinforcement.

$E_s$  = modulus of elasticity of steel in tension or compression  
(= 30,000,000 p.s.i.).

$E_c$  = modulus of elasticity of concrete in compression.

$$n = \frac{E_s}{E_c}$$

$M$  = bending moment or moment of resistance in general.

$A_s$  = effective area of reinforcement in tension in beams and slabs.

$A_s'$  = effective area of reinforcement in compression in beams and slabs.

$b$  = width of rectangular beam or width of flange of T-beam.

$d$  = depth from compression surface of beam or slab to center of longitudinal tensile reinforcement.

$d'$  = depth from compressive surface of beam to center of longitudinal compressive reinforcement.

$b'$  = width of web in T-beams.

$t$  = thickness of flange in T-beams.

$k$  = ratio of depth of neutral axis to depth  $d$ .

$j$  = ratio of lever arm of resisting couple to depth  $d$ .

$p$  = ratio of effective area of tensile reinforcement to effective area of concrete in beams or slabs =  $A_s \div bd$ .

$p'$  = ratio of effective area of compressive reinforcement to effective area of beam.

$V$  = total shear.

$v$  = shearing unit stress.

$v_c$  = shearing unit stress permitted on the concrete of the web; the value depending on the anchorage of the longitudinal reinforcement.

$$v' = v - v_c.$$

$\Sigma o$  = sum of perimeters of a group of bars.

$u$  = bond unit stress.

$l$  = clear span length of beam or slab.

$s$  = spacing of vertical stirrups, measured perpendicular to the direction of the stirrup.

$a$  = distance along the length of beam in which web reinforcement is required, measured from the face of support.

$w$  = uniformly distributed load per unit of length of beam or slab.

$P$  = concentrated load.



## 1. Rectangular Beams and Slabs

$$k = \sqrt{2pn + (pn)^2} - pn = \frac{1}{1 + \frac{f_s}{nf_c}}$$

$$j = 1 - \frac{k}{3}$$

$$p = \frac{A_s}{bd} = \frac{\frac{1}{2}}{\frac{f_s}{f_c} \left( \frac{f_s}{nf_c} + 1 \right)} = \frac{f_c k}{2f_s}$$

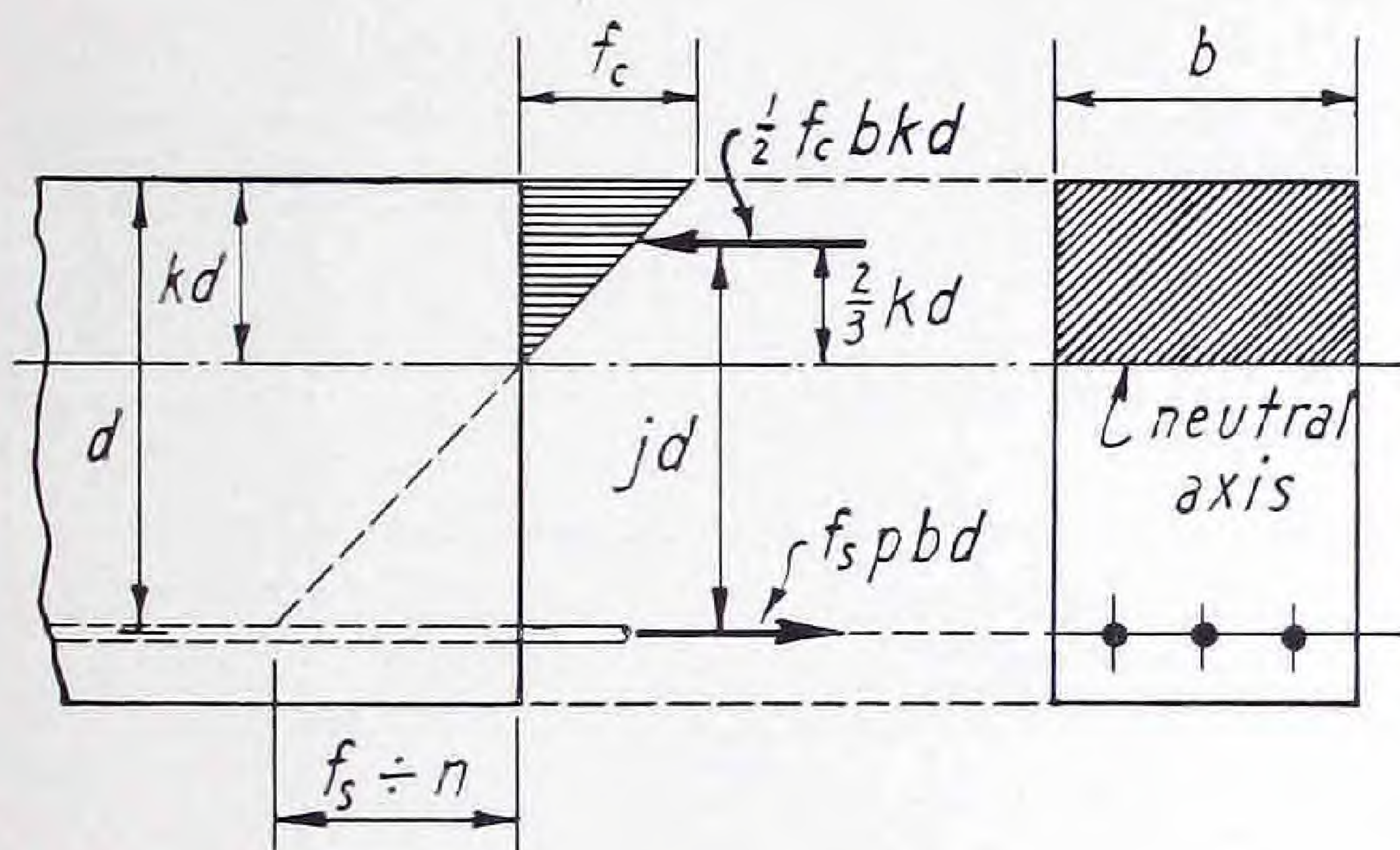


Fig. 2

$$K = \frac{1}{2} f_c k j \text{ or } p f_s j$$

$$M = \frac{1}{2} f_c k j b d^2 = K b d^2 \text{ or } b d^2 = \frac{2M}{f_c k j} = \frac{M}{K}$$

$$M = p f_s j b d^2 \text{ or } b d^2 = \frac{M}{p f_s j}$$

$$A_s = \frac{M}{f_s j d} \text{ or } f_s = \frac{M}{A_s j d}$$

$$f_c = \frac{2f_s p}{k} \text{ or } \frac{f_s k}{n(1 - k)} \text{ or } \frac{2M}{k j b d^2}$$



## 2. Rectangular Beams Reinforced for Compression

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p + p')^2} - n(p + p')$$

$$z = \frac{\frac{1}{3}k^3d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)}$$

$$jd = d - z$$

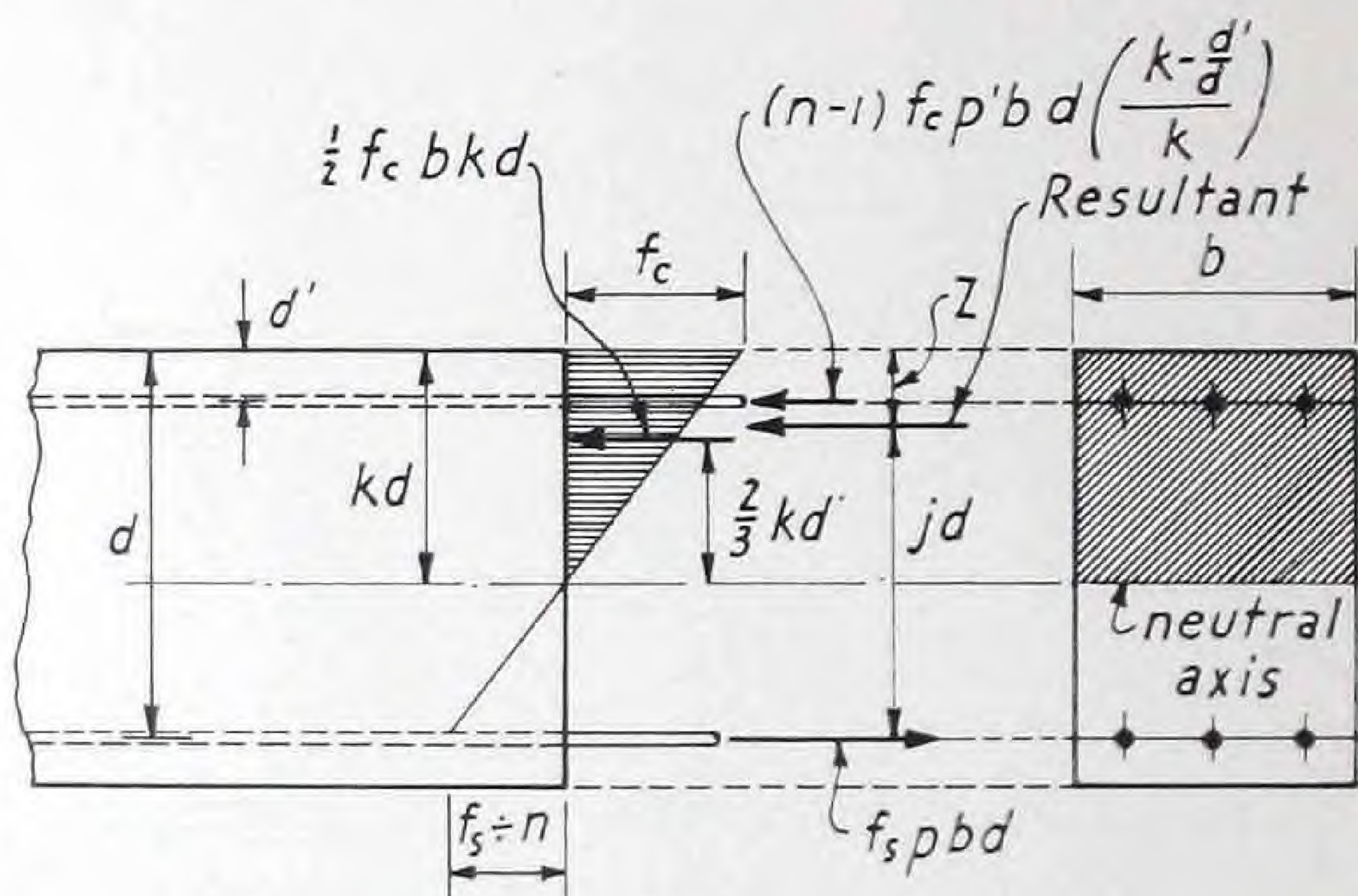


Fig. 3

$$f_c = \frac{6M}{bd^2 \left[ 3k - k^2 + \frac{6p'n}{k} \left( k - \frac{d'}{d} \right) \left( 1 - \frac{d'}{d} \right) \right]}$$

$$f_s = \frac{M}{p j b d^2} = n f_c \frac{1 - k}{k} \quad f'_s = n f_c \frac{k - \frac{d'}{d}}{k}$$

$$A_s = \frac{M}{f_s j d} \text{ or } p b d$$

$$A_{s'} = p' b d$$

$$M = K b d^2$$

$$d = \sqrt{\frac{M}{K b}}$$

$$p = \frac{f_c}{f_s} \left[ \frac{k}{2} + \frac{n-1}{k} \left( k - \frac{d'}{d} \right) p' \right]$$

$$K = f_c \left( \frac{k}{2} - \frac{k^2}{6} + \frac{n-1}{k} p' \left[ (1-k) \left( k - \frac{d'}{d} \right) + \left( k - \frac{d'}{d} \right)^2 \right] \right)$$



### 3. T-Beams

*Case I.* When the neutral axis lies in the flange, use the formulas for rectangular beams.

*Case II.* When the neutral axis lies in the web. The amount of compression in the web is commonly small compared with that in the flange and is neglected in the following formulas:

$$k = \frac{1}{1 + \frac{f_s}{nf_c}}$$

$$k = \frac{pn + \frac{1}{2}\left(\frac{t}{d}\right)^2}{pn + \frac{t}{d}}$$

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt}$$

$$d - jd = \frac{3kd - 2t}{2kd - t} \times \frac{t}{3}$$

$$j = \frac{6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 + \left(\frac{t}{d}\right)^3\left(\frac{1}{2pn}\right)}{6 - 3\left(\frac{t}{d}\right)}, \text{ or } j = 1 - \frac{t}{d} \left( \frac{3k - 2t/d}{6k - 3t/d} \right)$$

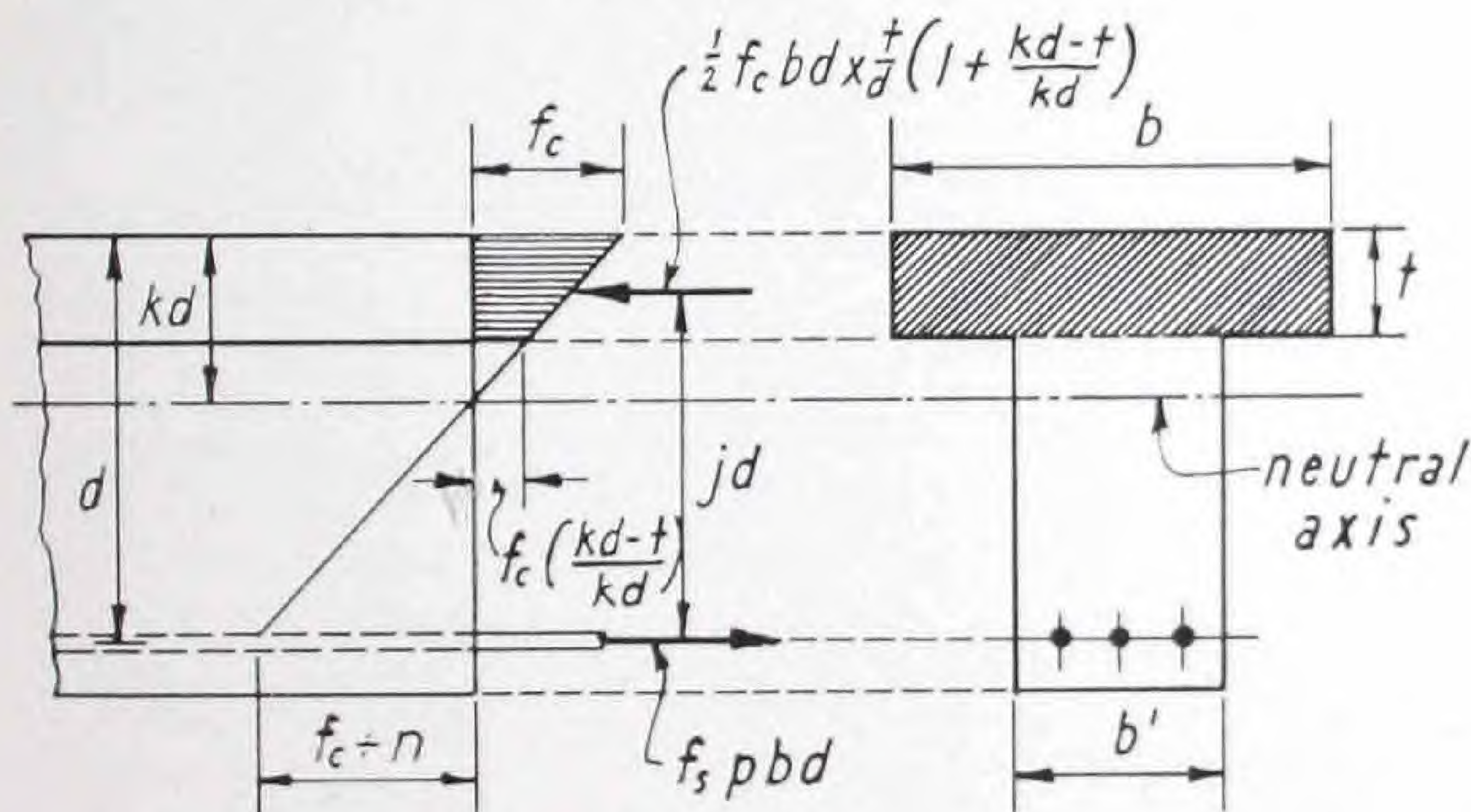


Fig. 4

$$f_s = \frac{M}{A_s jd} = \frac{M}{p j b d^2}$$

$$f_c = \frac{f_s k}{n(1 - k)}$$

$$M = f_s A_s jd$$

$$M = f_c \left( 1 - \frac{t}{2kd} \right) b t \times jd$$

### 4. Shear and Web Reinforcement

$$v = \frac{V}{b j d} \text{ (for rectangular beams)}$$



$$v = \frac{V}{b'jd} \text{ (for T-beams)}$$

$$a = \frac{l}{2} \left( \frac{v'}{v} \right)$$

### 5. Bond

$$u = \frac{V}{\Sigma ojd} = \frac{vb}{\Sigma o} \text{ or } \Sigma o = \frac{V}{ujd}$$

### Illustrative Examples

Computations for typical problems are given, using the tables in Section III and design data in this section as a basis, and following through to an estimate of the material quantities for each system. The computations are made by slide-rule. It is urged that the designer familiarize himself with each step, in order that he may learn the correct use of the tables.

#### Problem 1

Given an interior panel 18 ft. by 24 ft. with the following superimposed load:

Partitions	= 20 p.s.f.
Floor Finish	= 12 p.s.f.
Live Load	= 50 p.s.f.
Total	<u>82 p.s.f.</u>

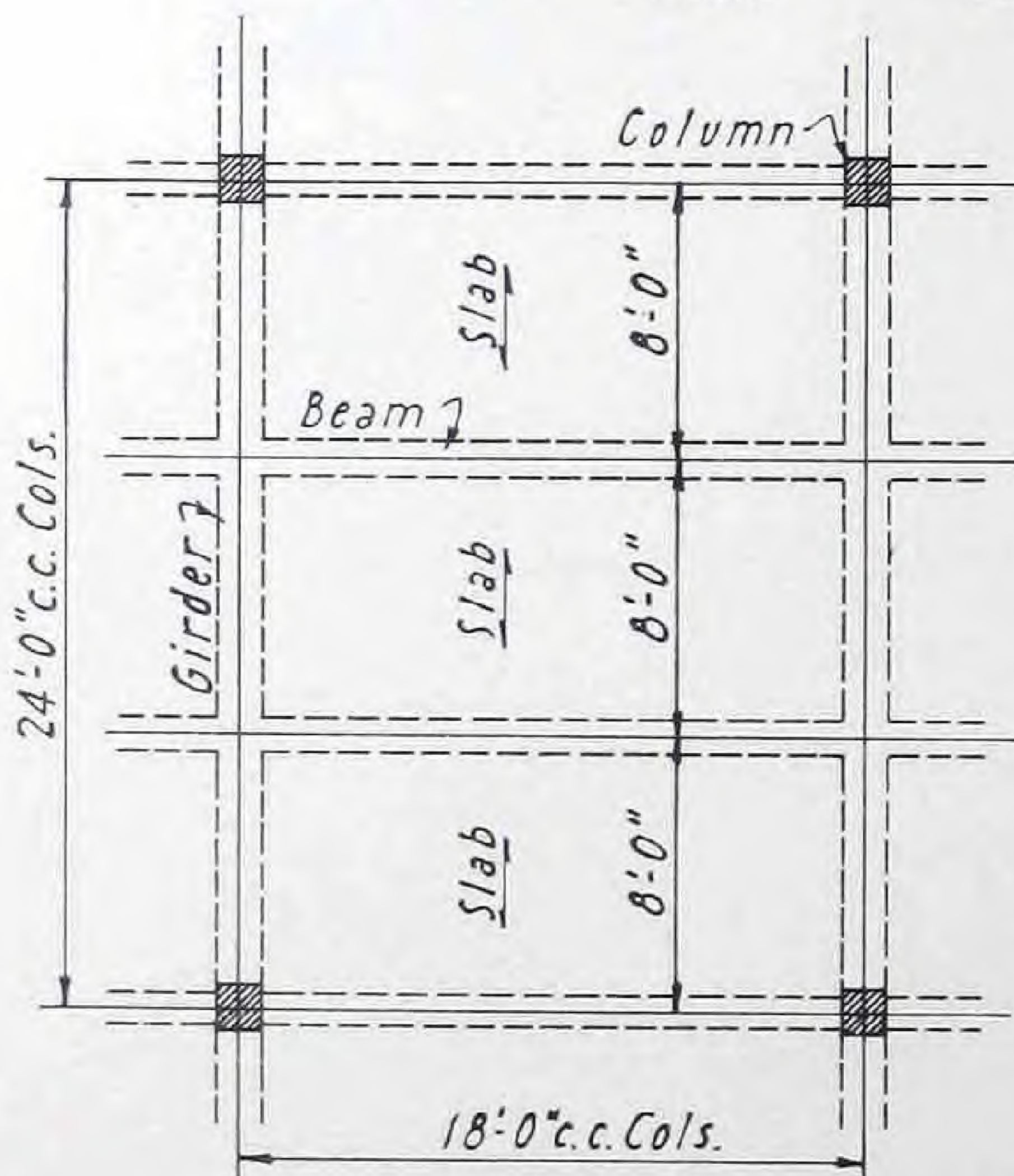


Fig. 5

If ceilings are plastered the weight of lath and plaster should be included in the total floor load.

Design a solid one-way slab, using intermediate beams framing into girders on column centers, as shown in Figure 5.

Clear slab span equals 8 ft. less width of beam, say 7 ft. Refer to Table No. 7. Down the column for spans of 7 ft. and opposite the line denoting an interior span, it will be found that the nearest (heavier) load is 98 p.s.f. This requires a slab 3 in. thick. However, in order to provide adequate



space for conduits and reinforcement a  $3\frac{1}{2}$  in. thick slab will be used for which the allowable safe load is 167 p.s.f.

Use 3½-in. slab. This slab weighs 44 p.s.f. making a total dead and live load of  $82 + 44 = 126$  p.s.f.

Then the area of steel required in a strip of slab one foot wide =

$$\frac{126}{167 + 44} \times 0.23 \text{ sq. in.} = 0.14 \text{ sq. in.}$$

According to Table No. 6,  $\frac{3}{8}$ -in. rd. bars spaced  $8\frac{1}{2}$  inches on centers satisfies this requirement, but for bond  $\Sigma o$  must equal

$$\frac{126 \times \frac{1}{2} (8.00 - 0.84)}{100 \times \frac{7}{8} (3.50 - 0.75 - \frac{1}{2} \times 0.375)} = 2.02 \text{ in.}$$

Use  $\frac{3}{8}$ -in. rd. bars at 7-in. o.c.\* Bend alternate bars over the beams and continue them in the top of the slab for a distance of one-fourth of the next span length (or in this case 2 ft. beyond the center line of the beam) on each side, as shown in Figure 6.

Reinforcement for temperature and shrinkage at right angles to main steel is also required. The spacing shall not exceed five times the depth of the slab or 18-in. The area of steel shall not be less than  $0.002\ bd$  with deformed bars or  $0.0025\ bd$  with plain bars. Use deformed bars.  $A_s = 0.002 \times 3\frac{1}{2} \times 12 = 0.084$  sq. in. Use  $\frac{3}{8}$ -in. rd. bars spaced 15 in. o.c. The maximum allowable spacing is not exceeded.

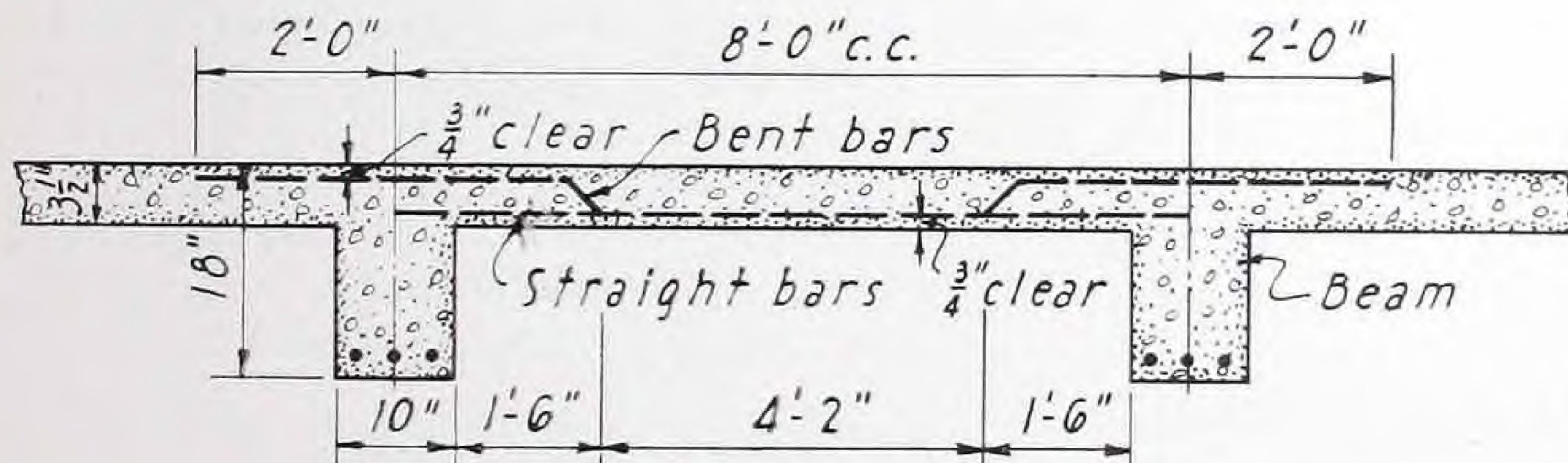


Fig. 6

## Design of Beams

Each beam carries a load of  $8(82 + 44) = 1008$  lb. per lin. ft. Clear span = say 17 ft. Referring to Table No. 16 (Safe load table of T-beams, interior spans), it will be found that a beam 10 in. by 18 in. is satisfactory. This beam can carry a load of 1200 lb. per lin. ft.

Use 10-in. by 18-in. beam. The stem of the beam weighs 151 lb. per lin. ft., making a total load on the beam of  $(1008 + 151) = 1159$  lb. per ft.

\*o.c.—spacing on centers of bars.



$$A_s = \frac{1159}{(1200 + 151)} \times 1.46 \text{ sq. in.} = 1.25 \text{ sq. in.}$$

Use one  $\frac{3}{4}$ -in. rd. bar, straight; two  $\frac{3}{4}$ -in. rd. bars, bent.

The total end shear equals half the entire load carried by the beam on its clear span. Assuming the width of the girders to be 16 inches, the clear span of the beam is 16 ft. 8 in.

$$\text{Then } V = \frac{16.67}{2} (1159) = 9650 \text{ lb. and the unit end shear}$$

$$v = \frac{V}{bjd} = \frac{9650}{10 \times 0.875^* \times 16} = 69 \text{ p.s.i.}$$

As this value exceeds the maximum allowable shearing unit stress of 40 p.s.i., take care of the excess by means of web reinforcement, such as stirrups. For their design, refer to the "Diagram for Stirrup Spacing," page 69.

Then  $v' = 69 - 40 = 29$  p.s.i. and  $v'b = 29 \times 10 = 290$ . The distance  $a$  over which stirrups are required  $= \frac{l}{2} \left( \frac{v'}{v} \right) = \frac{16.67}{2} \times \frac{29}{69} = 3.50$  ft. By code requirements, the maximum permissible spacing of stirrups  $= \frac{3}{4}d^{**} = 0.75 \times 16 = 12$  in.

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Holding a straight edge on the chart for  $\frac{3}{8}$ -in. rd. U-stirrups, so that it intersects the vertical line at 290 and the horizontal line at 3.50 ft., the spacing is read off at the intersections of the straight edge and the heavy vertical lines. In this case there are no intersections with lines indicating a spacing of 12 or less, so that stirrups are used at the maximum spacing of 12 inches up to a point equal to  $a$ . In other words, the spacing is  $S = 12, 12, 12$  in. Thus three stirrups are required at each end of the beam, or a total of six for each beam.

The maximum bond stress should not exceed 100 p.s.i.

$$u = \frac{vb}{\Sigma o} = \frac{69 \times 10}{2.36 \times 4} = 73 \text{ p.s.i.}$$

## Design of Girder

Clear span equals 24 ft. less width of column, assumed 16 in., or 22.67 ft. The girder, in addition to its own weight which acts as a uniform load, must support two concentrated loads due to the reactions of the intermediate beams. The load from each beam  $= \frac{18}{2} \left[ 8(82 + 44) \right] + \frac{16.67}{2} \times 151 = 9070 + 1260 = 10330$  lb. or a total concentrated load at the third-points of the girder  $= 2 \times 10330 = 20660$  lb.

\*See Table No. 3 for exact values of  $j$ . For practical purposes  $j$  is assumed as  $\frac{1}{4}$  or 0.875.

\*\*All examples of stirrup design have been based upon a maximum spacing of  $\frac{3}{4}d$  and a stress ( $f_s$ ) of 16000 p.s.i. The 1936 ACI Code, adopted after these examples were set in type, permits a maximum spacing of  $\frac{1}{2}d$  and a working stress of  $f_s = 20000$  p.s.i. To design for a 20000 p.s.i. working stress simply use the proper scale of ordinates in the diagrams on page 69.



Assume the weight of the girder = 375 lb. per lin. ft., uniformly distributed.

Referring to Table 1 and Fig. 1 of Moment Coefficients on Pages 14 and 15 to be used in this case, the equations for maximum moments (both the negative moment at the column and the positive moment at the center of the span) are as follows:

$$(1) -M = 0.211 \times 20660 \times 22.67 + \frac{1}{12} \times 375 \times 22.67^2 = 98,500 + 16,000 = 114,500 \text{ ft. lb.} = 1,375,000 \text{ in. lb.}$$

Assuming a width of girder stem of 16 in., the required effective depth =

$$\sqrt{\frac{M}{K^*b}} = \sqrt{\frac{1,375,000}{157 \times 16}} = \sqrt{548} = 23.5 \text{ (say 24 in., or a total depth of 26 in.)}$$

Use 16-in. by 26-in. girder.

$$A_s = \frac{M}{f_s j d} = \frac{1,375,000}{20,000 \times 0.875 \times 24} = 3.28 \text{ sq. in.}$$

$$(2) +M = 0.122 \times 20660 \times 22.67 + \frac{1}{12} \times 375 \times 22.67^2 = 57,100 + 16,000 = 73,100 \text{ ft. lb.} = 877,000 \text{ in. lb.}$$

Depth of compression flange,  $t = 3.5$  in., divided by the effective depth,  $d = 24$  in., equals  $\frac{t}{d} = 0.15$ , in which case the value of  $j$  is greater than  $\frac{7}{8}$  recommended for rectangular sections, but using  $j = \frac{7}{8}$  gives conservative results for  $A_s$ .

$$A_s = \frac{877,000}{20,000 \times 0.875 \times 24} = 2.09 \text{ sq. in.}$$

Use five  $\frac{3}{4}$ -in. rd. bars. Bend three bars over the supports and use one  $\frac{7}{8}$ -in. rd. extra bar, 12 ft. long in the top of the girder at the columns. This gives an area of 3.24 sq. in. at this point, where 3.28 sq. in. is required, which is sufficiently close.

The maximum end shear,  $V = 20660 + \frac{22.67}{2} \times 375 = 24,910 \text{ lb.}$

$$v = \frac{24,910}{16 \times 0.875 \times 24} = 74 \text{ p.s.i.} \quad v' = 74 - 40 = 34 \text{ p.s.i.}$$

$$v'b = 34 \times 16 = 544$$

The maximum shear at the first intermediate beam =  $20660 + 4.42 \times 375 = 22,320 \text{ lb.}$

$$v = \frac{22,320}{16 \times 0.875 \times 24} = 67 \text{ p.s.i.}$$

$$v' = 67 - 40 = 27 \text{ p.s.i.}$$

$$v'b = 27 \times 16 = 432$$

\*See Table 3 for values of  $K$ .



Use  $\frac{3}{8}$ -in. rd. U-stirrups. The maximum permissible spacing  $= 0.75d = 18$  in.

On the diagram of web reinforcement, page 69, place a mark at a height equal to 432 above a point 6.92\* on the  $a$  axis. Place a straight edge to connect this point and a point 544 on the  $v'b$  axis, and read the stirrup spacing:  $S = 9$  at 6, 3 at 9 in. or,  $2\frac{1}{4}$  stirrups per girder.

The bond stress at first intermediate beam,  $u = \frac{67 \times 16}{11.8} = 91$  p.s.i.

At the support,  $u = \frac{74 \times 16}{16.91} = 70$  p.s.i.

The depth of the girder designed above may be reduced by providing compressive reinforcement in the bottom of the girder at the supports. For this purpose advantage may be taken of the straight bars used as tensile reinforcement at the center of the span by extending the bars beyond the face of the support far enough to develop the stress in bond.

Assume that two  $\frac{3}{4}$ -in. rd. bars are brought from the other side of the support to provide, with the straight bars in the girder being designed, four  $\frac{3}{4}$ -in. rd. bars,  $A'_s = 1.76$  sq. in.

Determine the depth of the girder and the tensile reinforcement required by using the coefficients given in Table 4, and the formulas:  $d = \sqrt{\frac{M}{Kb}}$ ;  $A_s = pbd$ . The values necessary to enter this table are:  $d$  is assumed to be 21 in.;  $d' = 2.0$  in.;  $\frac{d'}{d} = 0.10$ ;  $p' = \frac{A'_s}{bd} = \frac{1.76}{16 \times 21} = 0.0052$ .

From the table, interpolating:  $K = 201$ ,  $p = 0.0115$

$d = \sqrt{\frac{1,375,000}{201 \times 16}} = 20.7$ , say 21 in.  $A_s = 0.0115 \times 16 \times 21 = 3.86$  in.

With this arrangement the depth may be reduced three inches but the tensile reinforcement must be increased about twenty per cent.

## Material Quantities

Concrete: Slab	$= 18 \times 24 \times 0.292$	$= 126.1$	cu. ft.
Beams	$= 3 \times 1.01 \times 16.67$	$= 50.6$	cu. ft.
Girder	$= 2.50 \times 22.67$	$= 56.7$	cu. ft.
Total		$= 233.4$	cu. ft. =
		$8.64$	cu. yd.

\*6.92 is distance in feet between face of column and face of first intermediate beam and is the space in which stirrups are required.



$$\begin{aligned}
 \text{Steel : Slabs} &= 3 \times \frac{12}{2 \times 7} \times 16.67 \times 8.0 \times .376 = 129 \text{ lb. (straight bars)} \\
 &3 \times \frac{12}{2 \times 7} \times 16.67 \times 12.0 \times .376 = 194 \text{ lb. (bent bars)} \\
 &22.67 \times \frac{12}{15} \times 19.5 \times 3.75 = 133 \text{ lb. (temp. bars)} \\
 &\quad \quad \quad \underline{456 \text{ lb.}}
 \end{aligned}$$

$$\begin{aligned}
 \text{Beams} &= 3 \times 1 \times 18 \times 1.502 = 81 \text{ lb. (straight bars)} \\
 &3 \times 2 \times 28 \times 1.502 = 252 \text{ lb. (bent bars)} \\
 &3 \times 6 \times 3.58 \times 0.376 = 24 \text{ lb. (stirrups)} \\
 &\quad \quad \quad \underline{357 \text{ lb.}}
 \end{aligned}$$

$$\begin{aligned}
 \text{Girder} &= 2 \times 24 \times 1.502 = 72 \text{ lb. (straight bars)} \\
 &3 \times 37.5 \times 1.502 = 169 \text{ lb. (bent bars)} \\
 &1 \times 12 \times 2.044 = 25 \text{ lb. (extra top bars)} \\
 &24 \times 5.42 \times 0.376 = 49 \text{ lb. (stirrups)} \\
 &\quad \quad \quad \underline{315 \text{ lb.}}
 \end{aligned}$$

$$\text{Total Steel} = 1128 \text{ lb.}$$

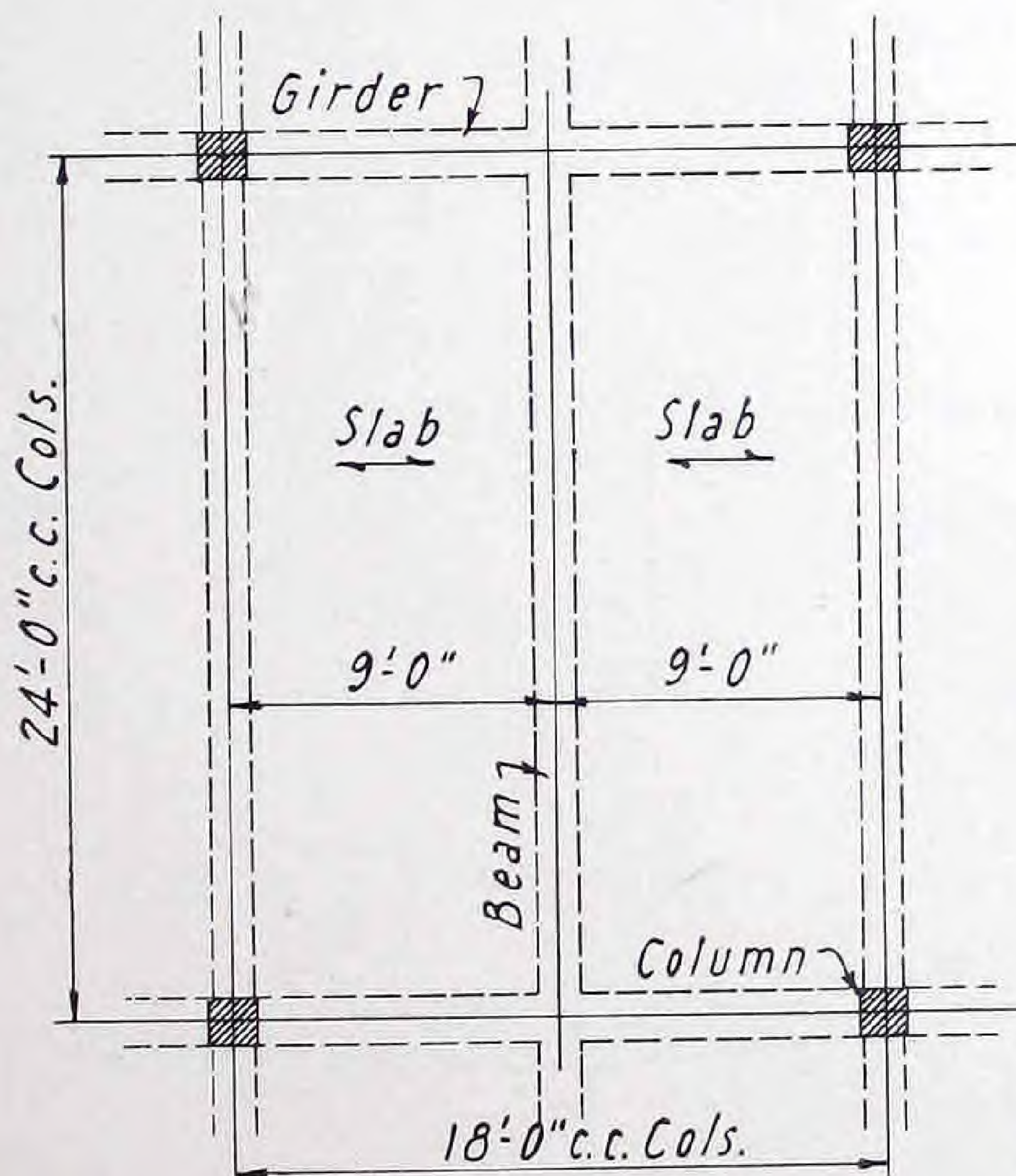


Fig. 7



## Problem 2

Given the same panel as in the preceding example, except that framing is as shown in Figure 7.

Clear span of slab is approximately 8 ft.

Total superimposed load = 82 p.s.f.

From Table No. 7: Use 3½-in. slab. Weight = 44 p.s.f.

$$A_s = \frac{82 + 44}{117 + 44} \times 0.23 = 0.18 \text{ sq. in. } \Sigma o = \frac{504}{100 \times \frac{7}{8} \times 256} = 2.24$$

Use ⅜-in. rd. bars at 6-in. o.c.

Bend alternate bars over supporting beams.

Temperature bars same as Problem 1; ⅜-in. rd. bars at 15-in. o.c.

## Design of Beams

Load = 9(82 + 44) = 1134 lb. per lin. ft.

Clear span = 22.67 ft., say 23 ft.

From Table No. 16: Use 12-in. by 22-in. beam. Web wt. = 232 lb. per lin. ft.

$$A_s = \frac{1134 + 232}{1150 + 232} \times 2.18 = 2.15 \text{ sq. in.}$$

Use five ¾-in. rd. bars, bend 3 bars in alternate beams and two bars in intermediate beams, thus providing five bars over all supports.

$$V = (9 \times 126 + 232) \frac{22.67}{2} = 15,500 \text{ lb.}$$

$$v = \frac{15,500}{12 \times 0.875 \times 20} = 74 \text{ p.s.i.}$$

$$v' = 74 - 40 = 34 \text{ p.s.i.}$$

$$v'b = 34 \times 12 = 408$$

$$a = \frac{22.67}{2} \times \frac{34}{74} = 5.20 \text{ ft.}$$

Use ⅜-in. rd. U-stirrups.

Spacing = 1 at 9; 1 at 12; 3 at 15 in. 10 required.

$$\text{Check for bond } u = \frac{74 \times 12}{11.80} = 75 \text{ p.s.i.}$$

## Design of Girders

$$\text{Concentrated load} = 2 \left( \frac{24}{2} \left[ 9(82 + 44) \right] + \frac{22.67}{2} \times 232 \right) = 32,460 \text{ lb.}$$

Assume uniform load (girder weight) = 270 lb. per lin. ft.

Clear span = 16.67 ft.

Assume  $b' = 14$  in.

$$(1) \quad -M = 0.119 \times 32,460 \times 16.67 + \frac{1}{12} \times 270 \times 16.67^2 = 70,600 \text{ ft. lb.} = 847,000 \text{ in. lb.}$$

$$\text{Required } d = \sqrt{\frac{847,000}{157 \times 14}} = \sqrt{385} = 19.6 \text{ (say 20 in.)}$$

Use 14-in. by 22-in. girder.



$$\text{At the supports, } A_s = \frac{847,000}{20,000 \times 0.875 \times 20} = 2.43 \text{ sq. in.}$$

$$(2) \quad +M = 0.130 \times 32,460 \times 16.67 + \frac{1}{12} \times 270 \times 16.67^2 = 70,300 + 6,200 = 76,500 \text{ ft. lb.} = 918,000 \text{ in. lb.}$$

$$A_s = \frac{918,000}{20,000 \times 0.875 \times 20} = 2.62 \text{ sq. in.}$$

$$\text{Use } \begin{cases} \text{Three } \frac{3}{4}\text{-in. rd. bars, straight.} \\ \text{Two } \frac{7}{8}\text{-in. rd. bars, bent.} \end{cases}$$

$$\text{Maximum End Shear} = \frac{32,460}{2} + \frac{16.67}{2} \times 270 = 16,230 + 2250 = 18,480 \text{ lb.}$$

$$v = \frac{18,480}{14 \times 0.875 \times 20} = 75 \text{ p.s.i.} \quad v' = 35 \text{ p.s.i.}$$

$$v'b = 35 \times 14 = 490$$

$$\text{Shear at center} = 16,230 \text{ lb.}$$

$$v = \frac{16,230}{14 \times 0.875 \times 20} = 66 \text{ p.s.i.} \quad v' = 26 \text{ p.s.i.}$$

$$v'b = 26 \times 14 = 364$$

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Maximum spacing =  $0.75 \times 20 = 15.0$  in.  
Spacing = 3 at 6, 9 at 9-in. 24 stirrups required.

$$\text{Bond at support, } u = \frac{75 \times 14}{4 \times 2.75} = 96 \text{ p.s.i.}$$

### Material Quantities

$$\begin{aligned} \text{Concrete: Slab} &= 18 \times 24 \times 0.292 = 126.1 \text{ cu. ft.} \\ \text{Beams} &= 2 \times 1.54 \times 22.67 = 69.8 \text{ cu. ft.} \\ \text{Girder} &= 1.80 \times 16.67 = 30.0 \text{ cu. ft.} \\ &\quad \underline{225.9 \text{ cu. ft.} = 8.35 \text{ cu. yd.}} \end{aligned}$$

$$\text{Steel: Slabs} = 2 \times \left( \frac{12}{2 \times 6} \right) \times 22.67 \times 9.0 \times 0.376 = 153 \text{ lb.}$$

(straight bars)

$$2 \times \left( \frac{12}{2 \times 6} \right) \times 22.67 \times 13.5 \times 0.376 = 230 \text{ lb.}$$

(bent bars)

$$16.67 \times \frac{12}{15} \times 25.5 \times 0.376 = 128 \text{ lb.}$$

(temp. bars)

$$\underline{511 \text{ lb.}}$$



$$\begin{aligned}
 \text{Beams} &= 2 \times 2 \times 24 \times 1.502 = 144 \text{ lb. (straight bars)} \\
 &2 \times 2\frac{1}{2} \times 37.25 \times 1.502 = 280 \text{ lb. (bent bars)} \\
 &2 \times 10 \times 4.58 \times 0.376 = 34 \text{ lb. (stirrups)} \\
 &\quad \quad \quad \underline{458 \text{ lb.}}
 \end{aligned}$$

$$\begin{aligned}
 \text{Girder} &= 3 \times 18 \times 1.502 = 81 \text{ lb. (straight bars)} \\
 &2 \times 28.25 \times 2.044 = 116 \text{ lb. (bent bars)} \\
 &24 \times 4.75 \times 0.376 = 44 \text{ lb. (stirrups)} \\
 &\quad \quad \quad \underline{241 \text{ lb.}}
 \end{aligned}$$

$$\text{Total steel} = 1210 \text{ lb.}$$

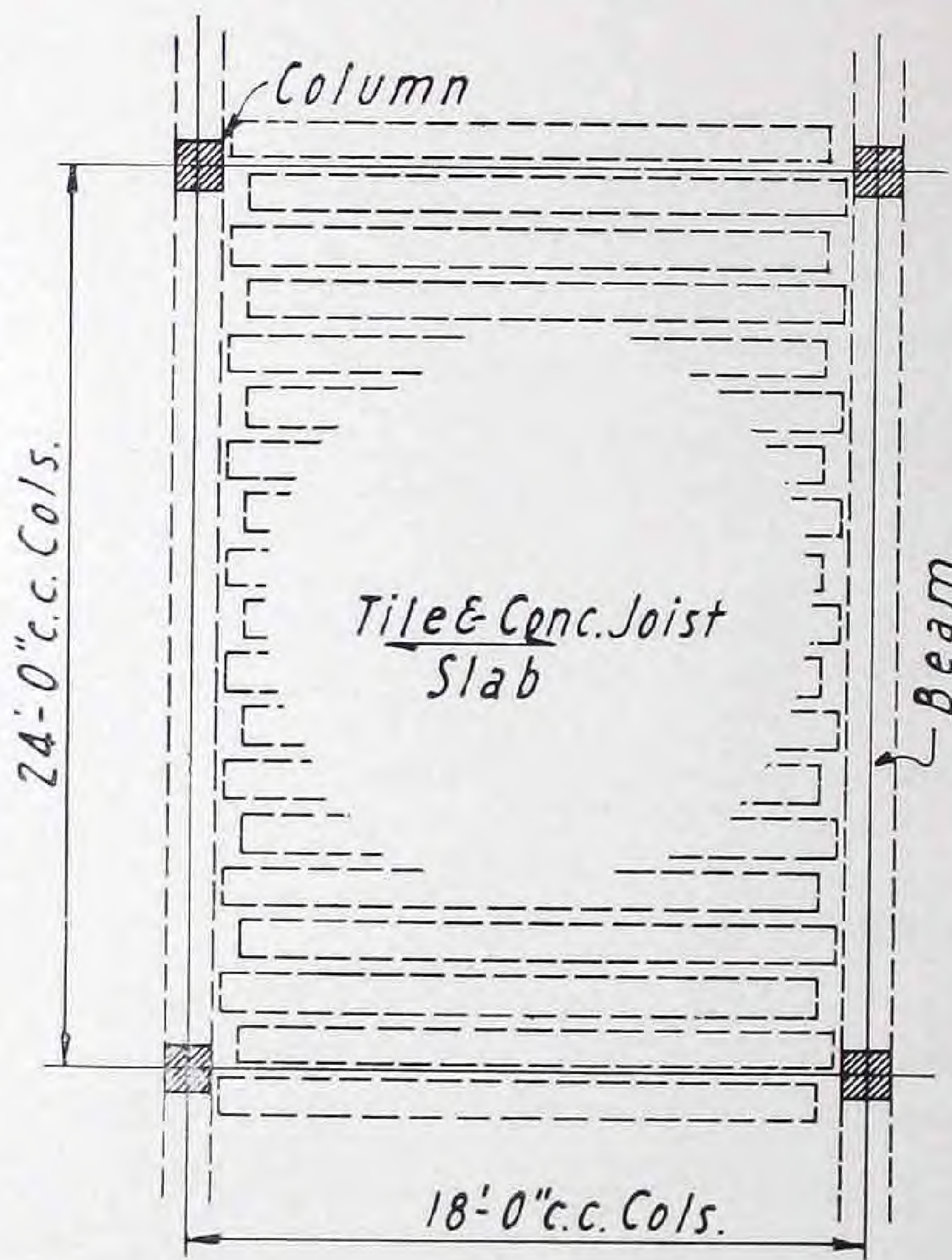


Fig. 8

### Problem 3

Given the same panel as in Problem 1, but framed as shown in Fig. 8. Design the panel, using one-way tile and concrete joist slab. Assume that beams are 16 in. wide. The clear span will be 16.67 ft., say 17 ft. The superimposed load is 82 p.s.f.

From Table No. 8, Use 6-in. tile and 3-in. topping, 6-in. joists — 18-in. o.c. The weight of this slab is 77 p.s.f.



$$\begin{aligned}\text{Required } A_s &= \frac{\text{Total Weight}}{\text{Load Coefficient}} \times \text{Steel Coefficient for } \frac{wl^2}{12} \\ &= \frac{82 + 77}{332} \times 1.05 = 0.50 \text{ sq. in. per joist}\end{aligned}$$

$$\text{Use } \left\{ \begin{array}{l} \text{One } \frac{1}{2}\text{-in. sq. bar, straight} \\ \text{One } \frac{1}{2}\text{-in. sq. bar, bent} \end{array} \right\} \text{ per joist.}$$

$$\text{Bond at support: } u = \frac{1980}{4 \times \frac{7}{8} \times 8} = 71 \text{ p.s.i.}$$

$$\text{Transverse reinforcement: } A_s = 0.002 \times 3 \times 12 = 0.07 \text{ sq. in.}$$

Use  $\frac{3}{8}$ -in. rd. bars at 15-in. o.c. 13 required.

Stagger tile joints in adjacent rows of tile as shown.

### Design of Beams

$$\text{Load: } 18(82 + 77) = 2860 \text{ lb. per lin. ft.}$$

Clear span: 22.67 ft. assuming 16-in. column.

From Table 16: Use 16-in. by 30-in. beam.

$$\text{Web wt.} = \frac{16 \times 21}{144} \times 150 + 2(112 - 77) = 408 \text{ lb. per ft.}$$

$$A_s = \frac{2860 + 408}{3230 + 408} \times 4.08 = 3.67 \text{ sq. in.}$$

$$\text{Use } \left\{ \begin{array}{l} \text{Three } \frac{7}{8}\text{-in. rd. bars, straight} \\ \text{Three } \frac{7}{8}\text{-in. rd. bars, bent} \end{array} \right\} A_s = 3.60 \text{ sq. in.}$$

$$V(2860 + 408) \times \frac{22.67}{2} = 36,900 \quad v = \frac{36,900}{16 \times 0.875 \times 28} = 94 \text{ p.s.i.}$$

$$v' = 94 - 40 = 54 \text{ p.s.i.} \quad v'b = 54 \times 16 = 865$$

$$a = \frac{54}{94} \times \frac{22.67}{2} = 6.50 \quad \text{Max. spacing} = 0.75 \times 28 = 21 \text{ in.}$$

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Spacing: 2 at 3, 4 at 6, 2 at 9, 2 at 12 in.  
20 required.

$$\text{Bond at support: } u = \frac{94 \times 16}{6 \times 2.75} = 92 \text{ p.s.i.}$$

### Material Quantities

Tile  $16 \times 16 = 256$  units of 6-in. Tile.

Concrete

$$\text{Slab} \quad \frac{18 \times 24 \times 9}{12} - \frac{256 \times 6}{12} = 196.0 \text{ cu. ft.}$$

$$\text{Beam} \quad \frac{16 \times 21}{144} \times 22.67 = 52.9 \text{ cu. ft.}$$

$$\overline{248.9} \text{ cu. ft.} = 9.23 \text{ cu. yd.}$$



*Steel*

Slab	$16 \times 18 \times 0.850$	= 245 lb. (straight)
	$16 \times 27.5 \times 0.850$	= 374 lb. (bent)
	$13 \times 25.5 \times 0.376$	= 124 lb. (temperature)
		<u>743 lb.</u>
Beam	$3 \times 24 \times 2.044$	= 148 lb. (straight)
	$3 \times 37.8 \times 2.044$	= 232 lb. (bent)
	$20 \times 6.08 \times 0.376$	= 45 lb. (stirrups)
		<u>425 lb.</u>

*Total Steel 1168 lb.*

**Problem 4**

Given the same panel as in Problem 1, framed as shown in Fig. 9. Design panel, using one-way tile and concrete joist slab.

The clear span of the slab, assuming 12-in. width of beams, is 11.0 ft. The superimposed load is 82 p.s.f.

From Table No. 8: Use 4-in. tile + 2-in. topping, 5-in. joists, 17 in. o.c. Weight of slab = 51 p.s.f.

$$A_s = \frac{133}{309} \times 0.62 = 0.27 \text{ sq. in. per joist}$$

$$\text{Use } \left\{ \begin{array}{l} \text{One } \frac{3}{8}\text{-in. rd. bar, straight} \\ \text{One } \frac{1}{2}\text{-in. rd. bar, bent} \end{array} \right\} \text{ per joist.}$$

$$\text{Bond at support: } u = \frac{1040}{3.14 \times \frac{7}{8} \times 5} = 76 \text{ p.s.i.}$$

Transverse or temperature reinforcement:  $A_s = 0.0025 \times 2.0 \times 12 = 0.06 \text{ sq. in.}$  Use  $\frac{1}{4}$ -in. rd. bars at 10 in. o.c. 26 required.

Stagger tile joints as shown in Fig. 9.

**Design of Beams**

Load:  $12 \times (82 + 51) = 1600 \text{ lb. per lin. ft.}$  Assuming a 16-in. width of girder, the clear span of the beam is 16.67 ft., say 17 ft. From Table No. 16: Use 12-in. by 20-in. beam.

$$\text{Web wt.} = \frac{12 \times 14}{144} \times 150 + 2(75 - 51) = 223 \text{ lb. per lin. ft.}$$

$$A_s = \frac{1600 + 223}{1860 + 223} \times 1.97 = 1.73 \text{ sq. in.}$$

Use four  $\frac{3}{4}$ -in. rd. bars, bend two.  $A_s = 1.76 \text{ sq. in.}$

$$V = (1600 + 223) \times \frac{16.67}{2} = 15,200 \text{ lb.} \quad v = \frac{15,200}{12 \times 0.875 \times 18} = 81 \text{ p.s.i.}$$

$$v' = 81 - 40 = 41 \text{ p.s.i.} \quad v'b = 41 \times 12 = 492$$

$$a = \frac{41}{81} \times \frac{16.67}{2} = 4.23 \text{ ft.} \quad \text{Max. spacing} = 0.75 \times 18 = 13.5 \text{ in.}$$



Use  $\frac{3}{8}$ -in. rd. U-stirrups, Spacing = 9, 12, 12, 12, 8 required per beam.

Bond at support:  $u = \frac{81 \times 12}{4 \times 2.36} = 103$  p.s.i. (Sufficiently close to allowable maximum)

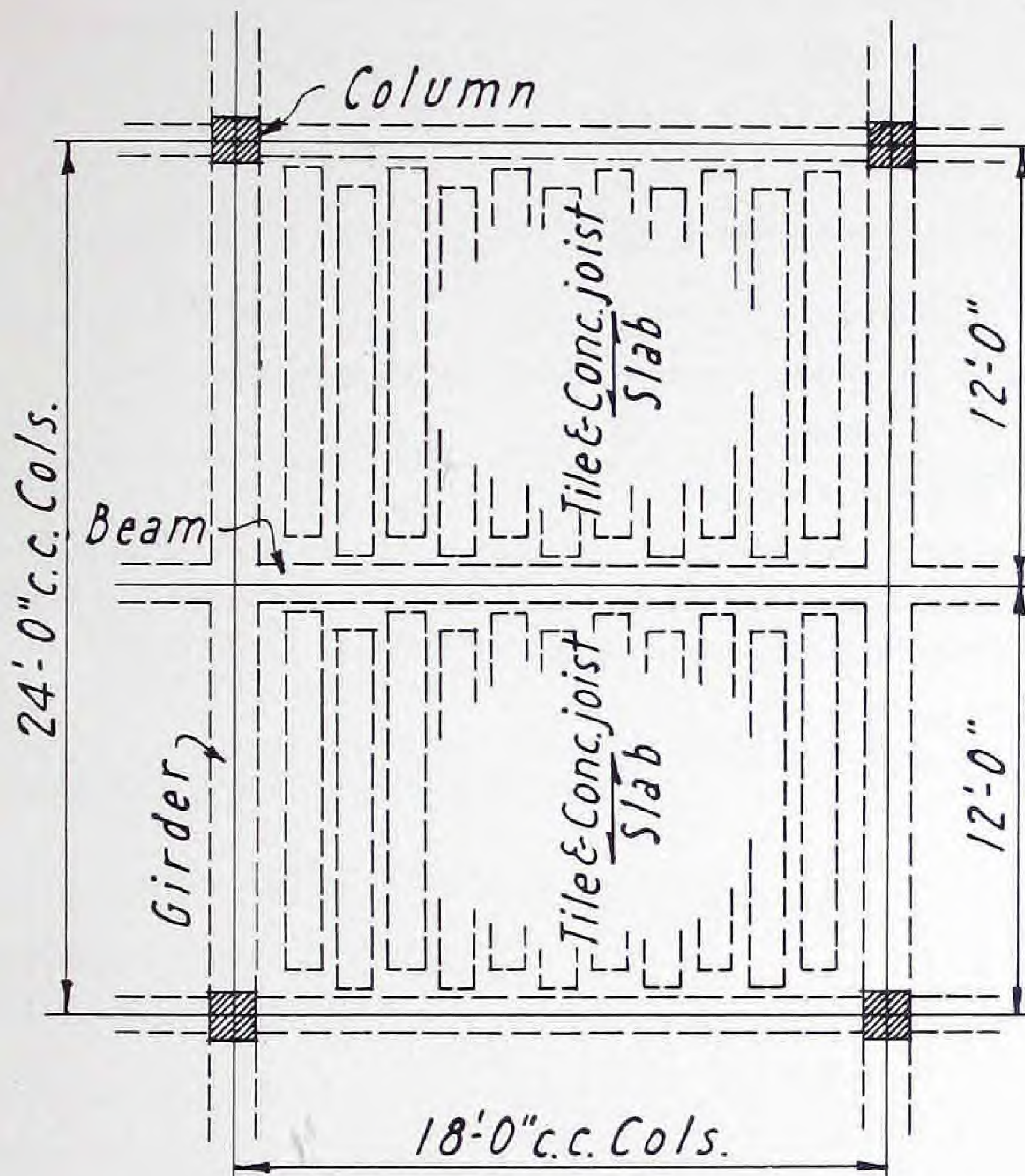


Fig. 9

### Design of Girder

Concentrated load:  $P = 2 \left( \frac{18}{2} \left[ 12 (82 + 51) \right] + \frac{16.67}{2} \times 223 \right) = 32,500$  lb.

Assumed girder weight: 350 lb. per lin. ft.

Clear span: 22.67 ft.

The coefficients of  $wl^2$  and  $Pl$  for moment are obtained from Table No. 1 and Fig. 1.

At support:  $-M = 0.119 \times 32,500 \times 22.67 + \frac{1}{12} \times 350 \times 22.67^2$   
 $= 87,800 + 15,000 = 102,800$  ft. lb.  $= 1,230,000$  in. lb.



Assume  $b' = 14$  in.

$$d = \sqrt{\frac{1,230,000}{157 \times 14}} = 23.6, \text{ say } 24 \text{ in.}$$

Use 14-in. by 26-in. girder.

$$A_s = \frac{1,230,000}{20,000 \times 0.875 \times 24} = 2.93 \text{ sq. in.}$$

At the center:

$$\begin{aligned} +M &= 0.130 \times 32,500 \times 22.67 + \frac{1}{12} \times 350 \times 22.67^2 \\ &= 95,800 + 15,000 = 110,800 \text{ ft. lb.} = 1,330,000 \text{ in. lb.} \end{aligned}$$

Using  $d = 24$  in. the required flange width is:

$$b = \frac{1,330,000}{131.3 \times 24 \times 24} = 17.6 \text{ in. Actual width is 34 in.}$$

$$A_s = \frac{1,330,000}{20,000 \times 0.875 \times 24} = 3.16 \text{ sq. in.}$$

Use four 1-in. rd. bars, bend two.

Shear at support:

$$V = \frac{32,500}{2} + \frac{22.67 \times 350}{2} = 20,210 \text{ lb.}$$

$$v = \frac{20,210}{14 \times 0.875 \times 24} = 69 \text{ p.s.i.} \quad v' = 69 - 40 = 29 \text{ p.s.i.}$$

$$v'b = 416$$

Shear at center:

$$V = \frac{16,250}{14 \times 0.875 \times 24} = 55 \text{ p.s.i.} \quad v' = 15 \text{ p.s.i.} \quad v'b = 210$$

Maximum stirrup spacing:  $\frac{3}{4} \times 24 = 18$  in.

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Spacing 5 at 9, 4 at 12, 2 at 15-in. 22 required.

$$\text{Bond at end: } u = \frac{69 \times 14}{4 \times 3.14} = 77 \text{ p.s.i.}$$

## Material Quantities

Tile  $2 \times 11 \times 10 = 220$  units of 4-in. Tile.

$$\text{Concrete Slab: } (18 \times 24) \frac{6}{12} - 220 \times \frac{4}{12} = 142.7 \text{ cu. ft.}$$

$$\text{Beams: } 2 \times 1.17 \times 16.83 = 39.5 \text{ cu. ft.}$$

$$\text{Girder: } 1.94 \times 22.67 = 44.0 \text{ cu. ft.}$$

$$\begin{aligned} &226.2 \text{ cu. ft.} = 8.40 \\ &\text{cu. yd.} \end{aligned}$$



<i>Steel</i>	Slab:	$2 \times 10 \times 12 \times 0.376$	= 90 lb. (straight)
		$2 \times 10 \times 18.3 \times 0.668$	= 244 lb. (bent)
		$26 \times 19 \times 0.168$	= 83 lb. (temperature)
			<u>417 lb.</u>
	Beams:	$2 \times 2 \times 18 \times 1.502$	= 108 lb. (straight)
		$2 \times 2 \times 28.1 \times 1.502$	= 169 lb. (bent)
		$2 \times 8 \times 4.1 \times 0.376$	= 25 lb. (stirrups)
			<u>302 lb.</u>
	Girder:	$2 \times 24 \times 2.670$	= 128 lb. (straight)
		$2 \times 37.6 \times 2.670$	= 200 lb. (bent)
		$22 \times 5.42 \times 0.376$	= 45 lb. (stirrups)
			<u>373 lb.</u>

*Total Steel = 1092 lb.*

### Problem 5

Given the same panel as in Problem 1, framed as shown in Fig. 10. Design panel using 20-in. metal pans and concrete joists. Use tapered pans. The clear span is approximately 17 ft., the superimposed load is 82 p.s.f.

From Table No. 10 use 6-in. pans + 2-in. topping, 6-in. joists — 26-in. o.c. Weight of slab is 48 p.s.f.

$$A_s = \frac{82 + 48}{244} \times 1.27 = 0.68 \text{ sq. in. per joist.}$$

$$Use \left\{ \begin{array}{l} \text{One } \frac{5}{8}\text{-in. rd. bar, straight} \\ \text{One } \frac{3}{4}\text{-in. rd. bar, bent} \end{array} \right\} \text{ per joist.}$$

$$\text{Bond at support: } u = \frac{2370}{4.72 \times \frac{7}{8} \times 6.88} = 83 \text{ p.s.i.}$$

Transverse reinforcement: The A. C. I. Code requires an area of at least 0.049 sq. in. per ft. width for floors carrying a live load not in excess of 50 p.s.f. Use  $\frac{1}{4}$ -in. rd. bars at 12-in. o.c. 17' required.

### Design of Beams

Load:  $18(82 + 48) = 2,340$  lb. per lin. ft. The clear span is 22.67 ft., say 23 ft.

From Table No. 16, Use 14-in. by 28-in. beam.

$$\text{Beam Weight: } \frac{14 \times 20}{144} \times 150 + 2(100 - 48) = 396 \text{ lb. per ft.}$$

$$A_s = \frac{2340 + 396}{2400 + 396} \times 3.31 = 3.23 \text{ sq. in.}$$

$$Use \left\{ \begin{array}{l} \text{Two 1-in. rd. bars, straight.} \\ \text{Three } \frac{7}{8}\text{-in. rd. bars, bent.} \end{array} \right.$$



$$V = (2340 + 396) \times \frac{22.67}{2} = 31,000 \text{ lb.}$$

$$v = \frac{31,000}{14 \times 0.875 \times 26} = 97 \text{ p.s.i.}$$

$$v' = 97 - 40 = 57 \text{ p.s.i.} \quad v'b = 57 \times 14 = 798$$

$$a = \frac{57}{97} \times \frac{22.67}{2} = 6.70 \text{ ft.}$$

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Maximum spacing = 19.5 in.

Spacing = 3, 5 at 6, 3 at 9, 1 at 15. 20 required.

$$\text{Bond at support: } u = \frac{97 \times 14}{6 \times 2.75} = 82 \text{ p.s.i.}$$

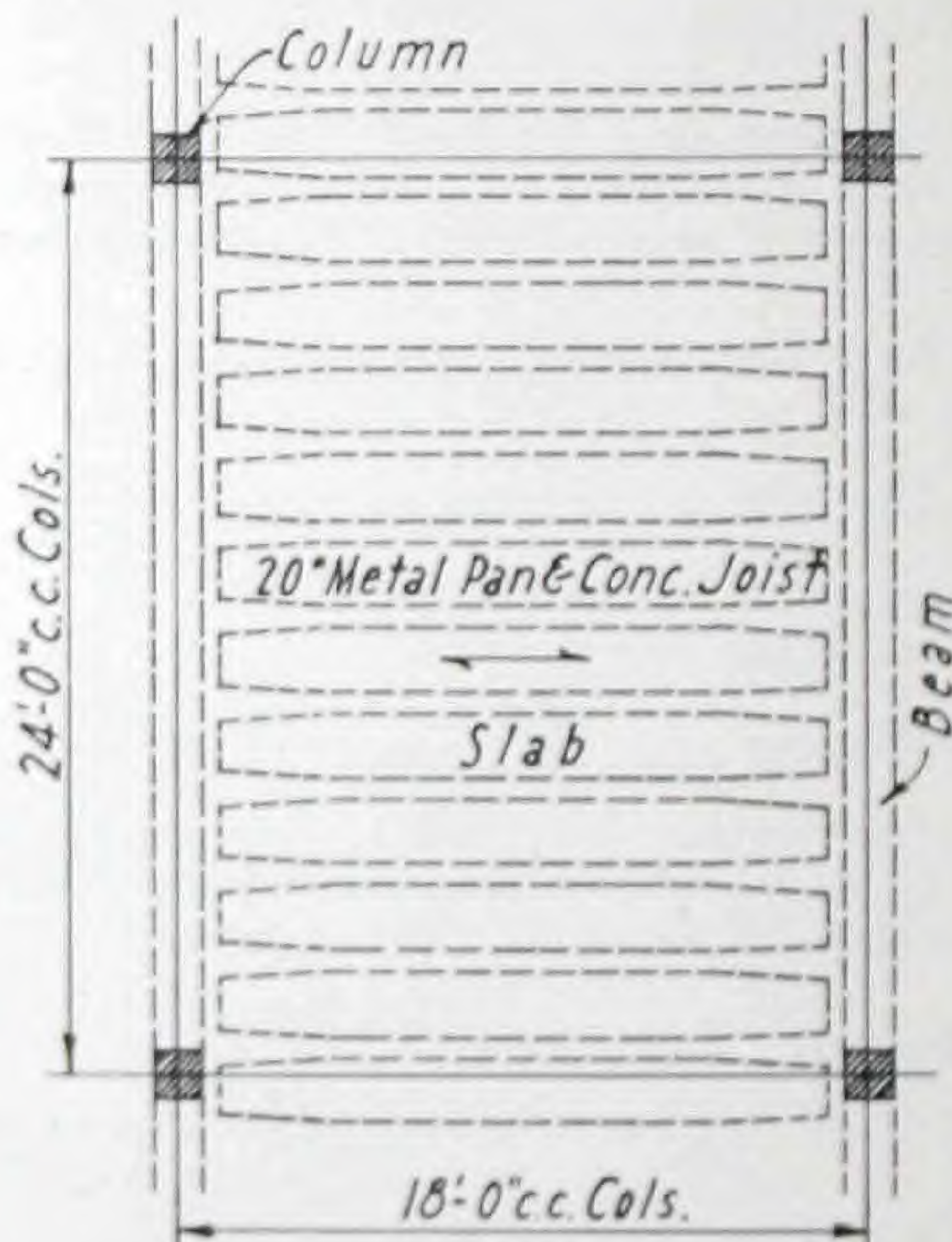


Fig. 10

## Material Quantities

### Concrete

$$\text{Slab: } 18 \times 24 \times \frac{8}{12} - \frac{12 \times 24}{26} \times \frac{6}{144} (10 \times 19.5 + 6 \times 17.5) = 150.0 \text{ cu. ft.}$$



$$\text{Beam: } 1.94 \times 22.67 = 44.2 \text{ cu. ft.}$$

$$194.2 \text{ cu. ft.} = 7.20 \text{ cu. yd.}$$

*Steel*

$$\text{Slab: } \frac{12 \times 24}{26} \times 18 \times 1.043 = 208 \text{ lb. (straight)}$$

$$\frac{12 \times 24}{26} \times 27.5 \times 1.502 = 458 \text{ lb. (bent)}$$

$$17 \times 25.0 \times 0.167 = 71 \text{ lb. (transverse)}$$

$$\underline{737 \text{ lb.}}$$

$$\text{Beam: } 2 \times 24 \times 2.670 = 128 \text{ lb. (straight)}$$

$$3 \times 37.7 \times 2.044 = 231 \text{ lb. (bent)}$$

$$20 \times 5.75 \times 0.376 = 43 \text{ lb. (stirrups)}$$

$$\underline{402 \text{ lb.}}$$

$$\text{Total Steel} = 1139 \text{ lb.}$$

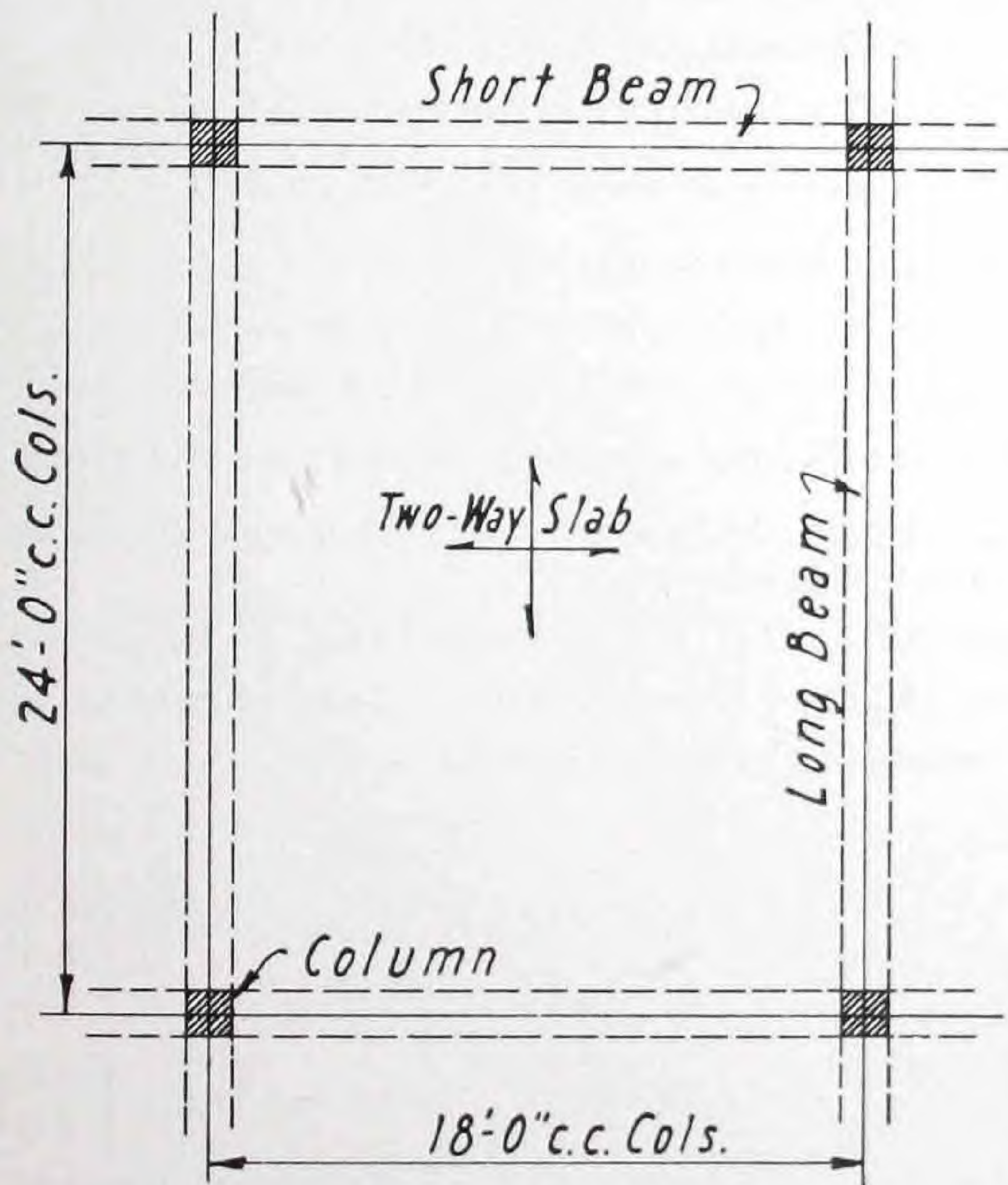


Fig. 11



### Problem 6\*

Given an interior panel, 18 ft. by 24 ft. between centerlines of beams as in Fig. 11, and a total superimposed load of 82 p.s.f. Determine size of slab and beams, amount of reinforcement, and quantities of materials required.

### Design of Slab

Assuming beam widths equal to 1 ft., the ratio of long to short clear spans is  $\frac{24 - 1}{18 - 1} = 1.35$ . For design of slab, use Table No. 19 (Ratio 1.4 to 1.0).

In column headed  $23.8 \times 17$  and for *Interior-Interior* condition of continuity, a  $5\frac{1}{2}$ -in. slab supporting 128 p.s.f. is seen to be sufficient. The  $5\frac{1}{2}$ -in. slab weighs 69 p.s.f.

Reinforcement given in table is:

Short way:  $\frac{1}{2}$ -in. rd. bars at  $5\frac{1}{2}$ -in. o.c. (area per linear foot = 0.44)

Long way:  $\frac{1}{2}$ -in. rd. bars at 9-in. o.c. (area per linear foot = 0.27)

With this reinforcement, the slab can carry

$128 + 69 = 197$  p.s.f.; but the actual load is  $82 + 69 = 151$  p.s.f.

Reduce the reinforcement above by multiplying it by  $\frac{151}{197} = 0.77$ :

Short way ( $0.77 \times 0.44 = 0.34$ ):  $\frac{1}{2}$ -in. rd. bars at 7-in. o.c.

Long way ( $0.77 \times 0.27 = 0.21$ ):  $\frac{1}{2}$ -in. rd. bars at 12-in. o.c.

This is the reinforcement required for center bands. The width of outer bands, adjacent to beams, is  $\frac{17}{4} = 4.25$  ft., and the width of center bands is  $23.0 - 2 \times 4.25 = 14.5$  ft. and  $17.0 - 2 \times 4.25 = 8.5$  ft. Reinforcement in outer bands is reduced to:

Short way ( $0.75 \times 0.77 \times 0.44 = 0.26$ ):  $\frac{1}{2}$ -in. rd. bars at 9-in. o.c.

Long way ( $0.75 \times 0.77 \times 0.27 = 0.16$ ):  $\frac{1}{2}$ -in. rd. bars at 14-in.\*\* o.c.

Total number of slab bars in a panel is:

Short way:

$$\frac{12 \times 14.5}{7} + \frac{12 \times 8.5}{9} = 24.9 + 11.3 = 36.2, \text{ say } 36 \text{ bars} \begin{cases} 18 \text{ straight.} \\ 18 \text{ bent.} \end{cases}$$

Long way:

$$\frac{12 \times 8.5}{12} + \frac{12 \times 8.5}{14} = 8.5 + 7.3 = 15.8, \text{ say } 15 \text{ bars} \begin{cases} 8 \text{ straight.} \\ 7 \text{ bent.} \end{cases}$$

\*Explanation and discussion pertaining to two-way concrete slabs are given on Pages 45 to 49.

\*\*14 in. is maximum spacing allowed. See explanation of Tables No. 17-22, on Pages 45 to 49.



## Design of Beams

Use beam load coefficients,  $(1 - er)$  and  $(1 - r)$  in Table No. 19 for *Interior-Interior* condition of continuity in the bottom lines: for bending, 0.835 (long beams), 0.440 (short beams); for shear, 0.733 (long beams), 0.267 (short beams). Column widths are taken as 16 in.

### Short Beam

Clear span:  $18.00 - 1.33 = 16.67$  ft. Use Table No. 16,  $l = 17$  ft.

Superimposed load for bending:

$$2 \times \frac{24}{2} \times 0.440 \times 151 = 1590 \text{ lb. per lin. ft.}$$

Use beam  $12 \times 20$  in. Weight of web:  $250 - 69 = 181$ , say 180 lb.

$$\text{From Table No. 16: } A_s = \frac{1590 + 180}{1860 + 180} \times 1.97 = 1.71 \text{ sq. in.}$$

$$\text{Use } \begin{cases} \text{Two } \frac{3}{4}\text{-in. rd. bars, straight.} \\ \text{Two } \frac{3}{4}\text{-in. rd. bars, bent.} \end{cases}$$

Superimposed load for shear:

$$2 \times \frac{24}{2} \times 0.267 \times 151 = 970 \text{ lb. per lin. ft.}$$

$$V = (970 + 180) \times \frac{16.67}{2} = 9,600 \text{ lb.}$$

$$v = \frac{9,600}{12 \times 0.875 \times 18} = 51 \text{ p.s.i.}$$

$$v' = 51 - 40 = 11 \text{ p.s.i.}$$

$$v'b = 11 \times 12 = 132$$

$$a = \frac{11}{51} \times \frac{16.67}{2} = 1.8 \text{ ft. Maximum spacing} = 0.75 \times 18 = 13.5 \text{ in.}$$

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Spacing = 2 at 12 in. 4 required.

$$\text{Bond at support: } u = 51 \times \frac{12}{4 \times 2.36} = 65 \text{ p.s.i.}$$

### Long Beam

Clear Span:  $24.00 - 1.33 = 22.67$  ft. Use Table No. 16,  $l = 23$  ft.

Superimposed load for bending:

$$2 \times \frac{18}{2} \times 0.835 \times 151 = 2270 \text{ lb. per lin. ft.}$$

Use beam  $14 \times 28$  in. Weight of web:  $408 - 69 = 339$ , say, 340 lb.

$$\text{From Table No. 16: } A_s = \frac{2270 + 340}{2400 + 340} \times 3.31 = 3.15 \text{ sq. in.}$$

$$\text{Use } \begin{cases} \text{Two 1-in. rd. bars, straight.} \\ \text{Two 1-in. rd. bars, bent.} \end{cases}$$



Superimposed load for shear:

$$2 \times \frac{18}{2} \times 0.733 \times 151 = 1990 \text{ lb. per lin. ft.}$$

$$V = (1990 + 340) \times \frac{22.67}{2} = 26,400 \text{ lb.}$$

$$v = \frac{26,400}{14 \times 0.875 \times 26} = 83 \text{ p.s.i.}$$

$$v' = 83 - 40 = 43 \text{ p.s.i.}$$

$$v'b = 43 \times 14 = 600$$

$$a = \frac{43}{83} \times \frac{22.67}{2} = 5.87. \text{ Maximum spacing} = 0.75 \times 26 = 19.5 \text{ in.}$$

Use  $\frac{3}{8}$ -in. rd. U-stirrups. Spacing 2 at 6, 2 at 9, 12, 15, 18 in. 14 required.

$$\text{Bond at support: } u = 83 \times \frac{14}{4 \times 3.14} = 93$$

## Material Quantities

### Concrete

$$\text{Slab: } 18 \times 24 \times 0.458 = 198.0 \text{ cu. ft.}$$

$$\text{Beams: } 1.21 \times 16.83 = 20.4 \text{ cu. ft.}$$

$$2.19 \times 24.00 = 52.6 \text{ cu. ft.}$$

$$\overline{271.0 \text{ cu. ft.}} = 10.03 \text{ cu. yd.}$$

### Steel

$$\text{Slab: } 18 \times 18.0 \times 0.668 = 217 \text{ lb. (straight)}$$

$$18 \times 27.3 \times 0.668 = 329 \text{ lb. (bent)}$$

$$8 \times 24.0 \times 0.668 = 128 \text{ lb. (straight)}$$

$$7 \times 36.3 \times 0.668 = 170 \text{ lb. (bent)}$$

$$\text{Beams: } 2 \times 18.0 \times 1.502 = 54 \text{ lb. (straight)}$$

$$2 \times 28.1 \times 1.502 = 84 \text{ lb. (bent)}$$

$$4 \times 4.08 \times 0.376 = 6 \text{ lb. (stirrups)}$$

$$2 \times 24.0 \times 2.670 = 128 \text{ lb. (straight)}$$

$$2 \times 37.7 \times 2.670 = 201 \text{ lb. (bent)}$$

$$14 \times 5.58 \times 0.376 = 29 \text{ lb. (stirrups)}$$

$$\text{Total} = \overline{1346 \text{ lb.}}$$

## Summary

A brief outline of design procedures has been presented in Problems 1 to 6 for various systems of concrete floor slabs. The problems also give quantities of material which are helpful in making comparisons prior to adopting the system best suited to a given set of conditions.

From the summary in Table No. 2, it is evident that for the fairly long spans used, and a light load, other things being equal, the ribbed floor with metal pans requires the least concrete, only slightly more reinforce-



TABLE No. 2—SUMMARY OF QUANTITIES

Problem No.	Type of Slab	Material Quantities		
		Concrete Cu. Yd.	Reinforcement Lb.	Filler Blocks No. and Size
1	One-way Solid Slab (1)	8.64	1128	
2	One-way Solid Slab (2)	8.35	1210	
3	One-way Tile Filler Ribbed Slab (1)	9.23	1168	256 of 6"
4	One-way Tile Filler Ribbed Slab (2)	8.40	1092	220 of 4"
5	One-way Metal Pan Filler Ribbed Slab	7.20	1139	
6	Two-way Solid Slab	10.03	1346	

ment than the type requiring the least steel and is the lightest construction because of the smaller quantity of concrete. This type of floor will in general prove most economical for the case considered. It can also be seen that the two-way solid slab is not well suited for this case. Had the superimposed load been much heavier, this type would be relatively more economical. A complete cost estimate must, of course, include the cost of forms.

SECTION III—STRUCTURAL DESIGN TABLES

TABLE No. 3—VALUES OF DESIGN CONSTANTS FOR VARIOUS COMBINATIONS OF STEEL AND CONCRETE STRESSES

$f_s$	16,000									
$n$	15					12		10		
$f_c$	650	700	750	800	900	1000	1125	1200	1350	
$p$	0.0077	0.0087	0.0097	0.0107	0.0129	0.0134	0.0161	0.0161	0.0193	
$k$	0.3786	0.3962	0.4128	0.4286	0.4576	0.4286	0.4576	0.4286	0.4576	
$j$	0.8738	0.8679	0.8624	0.8571	0.8475	0.8571	0.8475	0.8571	0.8475	
$K$	107.5	120.4	133.5	146.9	174.5	183.7	218.1	220.4	261.8	
$f_s$	18,000									
$n$	15					12		10		
$f_c$	650	700	750	800	900	1000	1125	1200	1350	
$p$	0.0063	0.0072	0.0080	0.0089	0.0107	0.0111	0.0134	0.0133	0.0161	
$k$	0.3514	0.3684	0.3846	0.4000	0.4288	0.4000	0.4286	0.4000	0.4286	
$j$	0.8829	0.8772	0.8718	0.8667	0.8571	0.8667	0.8571	0.8667	0.8571	
$K$	100.8	113.1	125.7	138.7	165.4	173.3	206.6	208.0	247.9	
$f_s$	20,000									
$n$	15					12		10		
$f_c$	650	700	750	800	900	1000	1125	1200	1350	
$p$	0.0053	0.0060	0.0068	0.0075	0.0091	0.0094	0.0113	0.0112	0.0136	
$k$	0.3277	0.3443	0.3600	0.3750	0.4030	0.3750	0.4030	0.3750	0.4030	
$j$	0.8908	0.8852	0.8800	0.8750	0.8657	0.8750	0.8657	0.8750	0.8657	
$K$	94.87	106.7	118.8	131.3	157.0	164.1	196.2	196.9	235.5	



TABLE No. 4—RECTANGULAR BEAMS WITH COMPRESSIVE REINFORCEMENT DESIGN VALUES OF  $p$  AND  $K$  FOR  $f_s = 20,000, f_c = 900, n = 15$

$p'$	$d'/d = 0.02$		$d'/d = 0.04$		$d'/d = 0.06$		$d'/d = 0.08$		$d'/d = 0.10$	
	$p$	$K$	$p$	$K$	$p$	$K$	$p$	$K$	$p$	$K$
0.002 . . .	0.0103	180	0.0102	179	0.0102	177	0.0101	176	0.0100	174
0.004 . . .	0.0115	204	0.0114	200	0.0112	197	0.0111	194	0.0110	191
0.006 . . .	0.0127	227	0.0125	222	0.0123	218	0.0121	213	0.0119	208
0.008 . . .	0.0139	251	0.0136	244	0.0134	238	0.0131	231	0.0129	225
0.010 . . .	0.0151	274	0.0148	266	0.0145	258	0.0141	250	0.0138	242
0.012 . . .	0.0163	298	0.0159	288	0.0155	278	0.0151	268	0.0148	259
0.014 . . .	0.0175	321	0.0170	310	0.0166	298	0.0161	287	0.0157	276
0.016 . . .	0.0187	345	0.0181	331	0.0177	319	0.0172	306	0.0167	293
0.018 . . .	0.0199	368	0.0193	353	0.0188	339	0.0182	324	0.0176	310
0.020 . . .	0.0211	392	0.0204	375	0.0198	359	0.0192	343	0.0186	328

$p'$	$d'/d = 0.12$		$d'/d = 0.14$		$d'/d = 0.16$		$d'/d = 0.18$		$d'/d = 0.20$	
	$p$	$K$	$p$	$K$	$p$	$K$	$p$	$K$	$p$	$K$
0.002 . . .	0.0100	173	0.0099	171	0.0099	170	0.0098	168	0.0097	167
0.004 . . .	0.0109	188	0.0107	185	0.0106	182	0.0105	180	0.0104	177
0.006 . . .	0.0117	204	0.0116	199	0.0114	195	0.0112	191	0.0110	187
0.008 . . .	0.0126	219	0.0124	214	0.0121	208	0.0119	203	0.0116	198
0.010 . . .	0.0135	235	0.0132	228	0.0129	221	0.0126	214	0.0123	208
0.012 . . .	0.0144	250	0.0140	242	0.0136	234	0.0133	226	0.0129	218
0.014 . . .	0.0153	266	0.0148	256	0.0144	246	0.0140	237	0.0135	228
0.016 . . .	0.0161	282	0.0157	270	0.0152	259	0.0147	249	0.0142	238
0.018 . . .	0.0170	297	0.0165	284	0.0159	272	0.0154	260	0.0148	248
0.020 . . .	0.0179	313	0.0173	298	0.0167	285	0.0161	271	0.0154	259

TABLE No. 5—SECTIONAL AREA IN SQ. IN. OF VARIOUS NUMBERS OF BARS

Number of Bars	Size . . .	$\frac{3}{8}"\phi$	$\frac{1}{2}"\phi$	$\frac{1}{2}"\square$	$\frac{5}{8}"\phi$	$\frac{3}{4}"\phi$	$\frac{7}{8}"\phi$	1" $\phi$	1" $\square$	1 $\frac{1}{8}"\square$	1 $\frac{1}{4}"\square$
	Wt. per Ft.	0.376	0.668	0.850	1.043	1.502	2.044	2.670	3.400	4.303	5.313
	Perimeter .	1.18	1.57	2.00	1.96	2.36	2.75	3.14	4.00	4.50	5.00
1 . . . . .		0.11	0.20	0.25	0.31	0.44	0.60	0.79	1.00	1.27	1.56
2 . . . . .		0.22	0.40	0.50	0.62	0.88	1.20	1.58	2.00	2.54	3.12
3 . . . . .		0.33	0.60	0.75	0.93	1.32	1.80	2.37	3.00	3.81	4.68
4 . . . . .		0.44	0.80	1.00	1.24	1.76	2.40	3.16	4.00	5.08	6.24
5 . . . . .		0.55	1.00	1.25	1.55	2.20	3.00	3.95	5.00	6.35	7.80
6 . . . . .		0.66	1.20	1.50	1.86	2.64	3.60	4.74	6.00	7.62	9.36
7 . . . . .		0.77	1.40	1.75	2.17	3.08	4.20	5.53	7.00	8.89	10.92
8 . . . . .		0.88	1.60	2.00	2.48	3.52	4.80	6.32	8.00	10.16	12.48
9 . . . . .		0.99	1.80	2.25	2.79	3.96	5.40	7.11	9.00	11.43	14.04
10 . . . . .		1.10	2.00	2.50	3.10	4.40	6.00	7.90	10.00	12.70	15.60
11 . . . . .		1.21	2.20	2.75	3.41	4.84	6.60	8.69	11.00	13.97	17.16
12 . . . . .		1.32	2.40	3.00	3.72	5.28	7.20	9.48	12.00	15.24	18.72
13 . . . . .		1.43	2.60	3.25	4.03	5.72	7.80	10.27	13.00	16.51	20.28
14 . . . . .		1.54	2.80	3.50	4.34	6.16	8.40	11.06	14.00	17.78	21.84
15 . . . . .		1.65	3.00	3.75	4.65	6.60	9.00	11.85	15.00	19.05	23.40



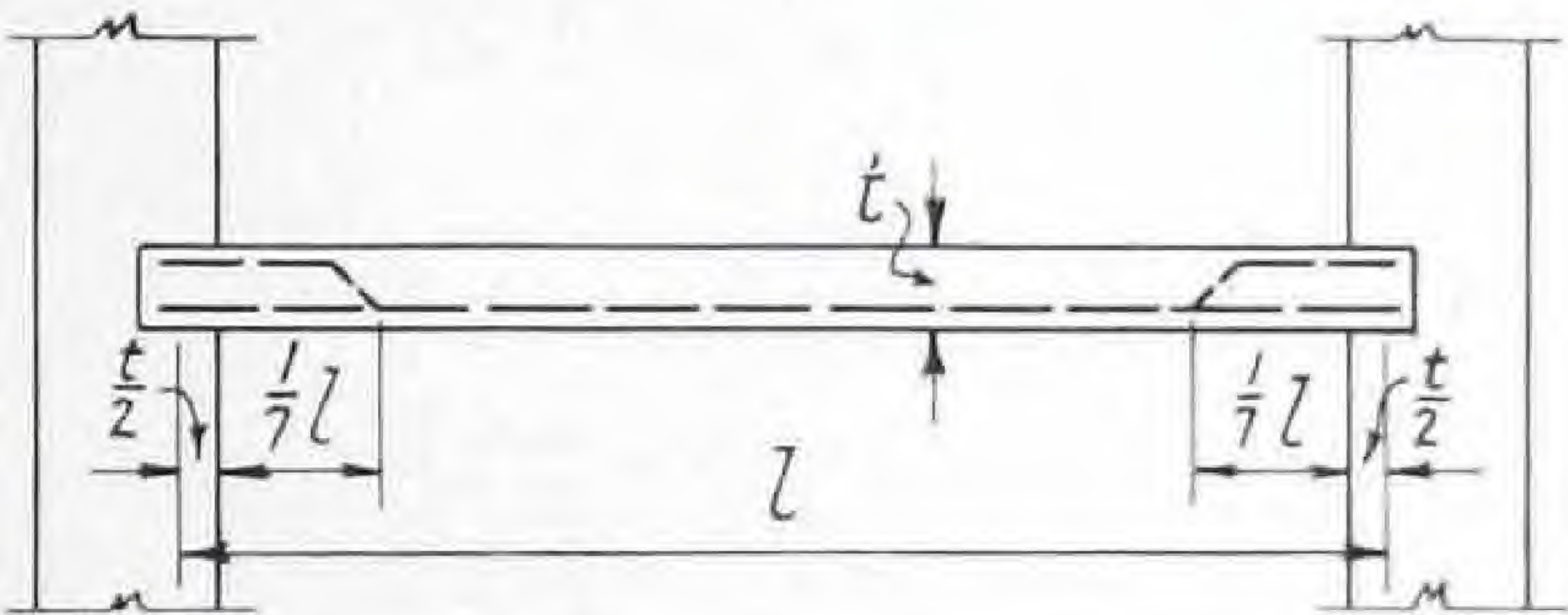
TABLE No. 6—AREA OF STEEL IN SQ. IN. PER LINEAR FOOT OF SLAB FOR RODS SPACED AT VARIOUS INTERVALS

Spacing c. to c. in Inches	$\frac{3}{8}"\phi$	$\frac{1}{2}"\phi$	$\frac{1}{2}"\square$	$\frac{5}{8}"\phi$	$\frac{3}{4}"\phi$	$\frac{7}{8}"\phi$	$1"\phi$	$1"\square$	$1\frac{1}{8}"\square$	$1\frac{1}{4}"\square$
3. . . . .	0.44	0.80	1.00	1.24	1.76	2.40	3.16	4.00	5.08	
3½. . . . .	0.38	0.69	0.86	1.06	1.51	2.06	2.71	3.43	4.35	5.35
4. . . . .	0.33	0.60	0.75	0.93	1.32	1.80	2.37	3.00	3.81	4.68
4½. . . . .	0.29	0.53	0.67	0.83	1.17	1.60	2.11	2.67	3.39	4.16
5. . . . .	0.26	0.48	0.60	0.74	1.06	1.44	1.90	2.40	3.05	3.74
5½. . . . .	0.24	0.44	0.55	0.68	0.96	1.31	1.72	2.18	2.77	3.40
6. . . . .	0.22	0.40	0.50	0.62	0.88	1.20	1.58	2.00	2.54	3.12
6½. . . . .	0.20	0.37	0.46	0.57	0.81	1.11	1.46	1.85	2.34	2.88
7. . . . .	0.19	0.34	0.43	0.53	0.75	1.03	1.35	1.71	2.18	2.67
7½. . . . .	0.18	0.32	0.40	0.50	0.70	0.96	1.26	1.60	2.03	2.50
8. . . . .	0.16	0.30	0.37	0.46	0.66	0.90	1.18	1.50	1.90	2.34
8½. . . . .	0.15	0.28	0.35	0.44	0.62	0.85	1.12	1.41	1.79	2.20
9. . . . .		0.27	0.33	0.41	0.59	0.80	1.05	1.33	1.69	2.08
9½. . . . .		0.25	0.32	0.39	0.56	0.76	1.00	1.26	1.60	1.97
10. . . . .		0.24	0.30	0.37	0.53	0.72	0.95	1.20	1.52	1.87

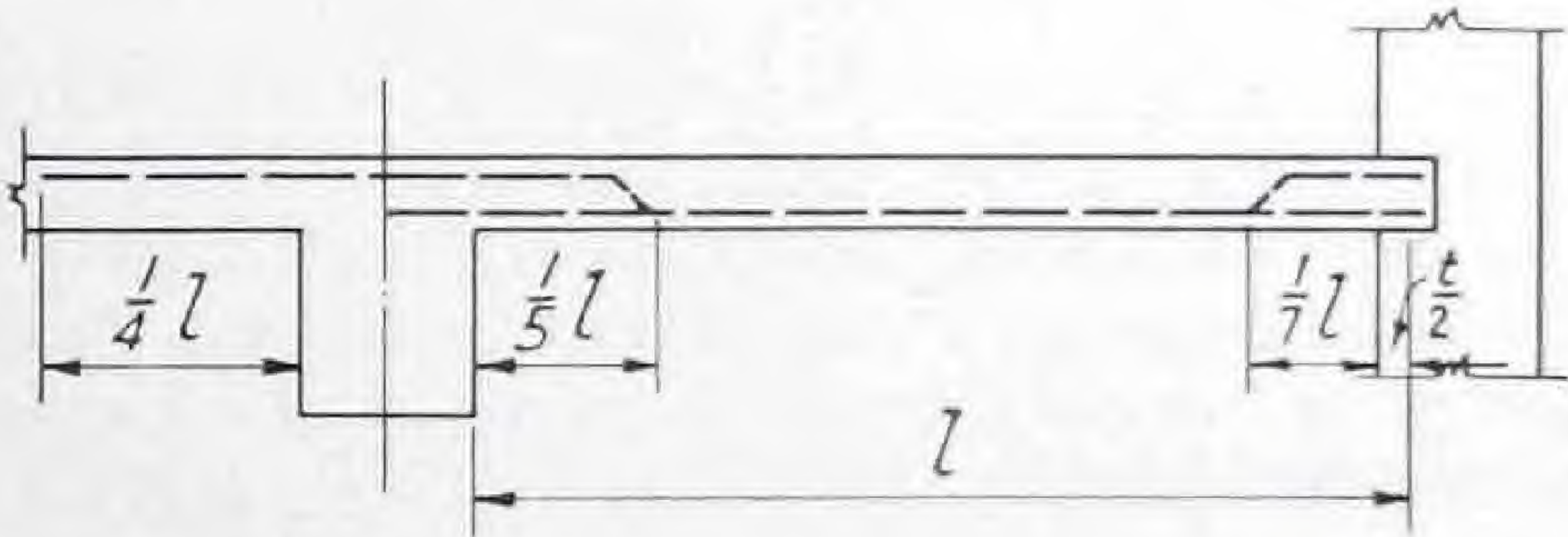
EXPLANATION OF TABLE No. 7

To determine the depth of slab for a given superimposed load, clear span and “condition of continuity,” read down the column for the given clear span and across from the given “condition of continuity” until a safe load

SIMPLE SPAN



END SPAN



INTERIOR SPAN

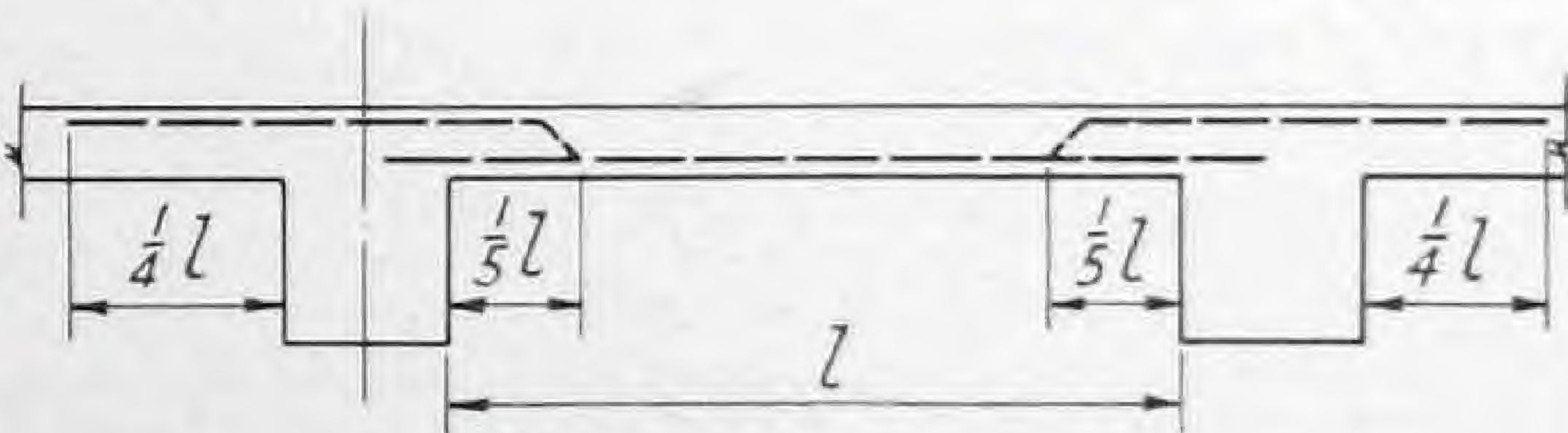


Fig. 12



is reached, which is equal to or greater than that required. The thickness of slab is shown in the first column of the table.

If the safe load is equal to the given load, use the size and number of bars shown in the fifth and sixth columns. If the safe load is larger than the given load, the steel area shown in the fourth column may be reduced by the proportion:

$$\frac{\text{given superimposed load} + \text{weight of slab}}{\text{safe superimposed load} + \text{weight of slab}}$$

The size and spacing of reinforcing steel may then be selected by use of Table 6, page 41. Problem 1, page 20, illustrates the use of Table 7.

Typical bar bending diagrams for simple, end and interior solid slab spans are shown in Fig. 12.

## EXPLANATION OF TABLES No. 8 TO 12

The tables of safe loads for ribbed slabs are based upon clear spans, assumed 1-ft. less than the center to center spans, for both moment and shear. In the tables for floors involving tile fillers, 1½-in. has been added to the joist widths when computing the shearing value of the joist. This is done in order to take into account the shells of the tile in contact with the joist. Such allowance can only be made when the joints in the tile are staggered.\*

The safe superimposed loads in Table 8 have been computed, taking into consideration the following weights of filler units (each unit 12-in. by 12-in. by the depth shown):

Depth of Tile In.	Weight per Unit Lbs.
4	16
6	22
8	30
10	36
12	40

If the weight of filler units used varies from that in the above table, the tabular values of safe superimposed loads must be corrected accordingly.

In order to design a ribbed slab having the clear span and the superimposed load in pounds per sq. ft. enter the proper table (depending on type of filler units) in the column for the given span; follow down the column until a safe load is reached equal or greater than the given load; read the slab dimensions and joist width in columns one and three. The area of steel required per joist is determined by the following formula:

$$A_s = \frac{\text{Design Load} + \text{Weight of Slab}}{\text{Load Coefficient}} \times \text{Steel Coefficient}$$

\*The shells of the tile are considered effective as compression area for negative moments near supports. It is also assumed in all joist slab tables that compressive steel at the interior support of two spans is equal to that required at midspan for positive moment and at interior supports of three or more spans the compressive steel is one-half of the positive reinforcement. The length of embedment of straight bottom bars shall be not less than 20 diameters beyond end of the joist ribs.



The *Load Coefficient* is found in the table just above the safe load value for the particular span and slab dimensions. The *Steel Coefficient* is found opposite the safe load value in columns 6, 7 or 8, depending upon the condition of continuity.

### Illustrative Problem

The following example illustrates the use of Tables 8 to 12, inclusive.

Design a tile and concrete joist slab, interior panel, having a clear span face to face of supporting beams of 16-ft. and carrying a superimposed load of 100 p.s.f. Enter Table 8, under span 16-ft., follow down the column to the first safe load value greater than 100 p.s.f.; namely, 107 p.s.f. In the first column it is found that an "8 + 2 Slab" is adequate using a 5-in. joist as indicated in Column 3. The weight of the slab, including the tile filler, is 75 p.s.f.

The load coefficient is 416 and the steel coefficient for the condition of continuity,  $\frac{wl^2}{12}$  is 0.95. The steel area required is then obtained from the formula for  $A_s$  given above.

$$A_s = \frac{(100 + 75)}{416} \times 0.95 = 0.40 \text{ sq. in.}$$

Table 5 shows two  $\frac{1}{2}$ -in. rd. bars have an area of 0.40 sq. in.

Typical bar bending diagrams for tile and metal pan filler and concrete joist slabs are shown in Fig. 13.

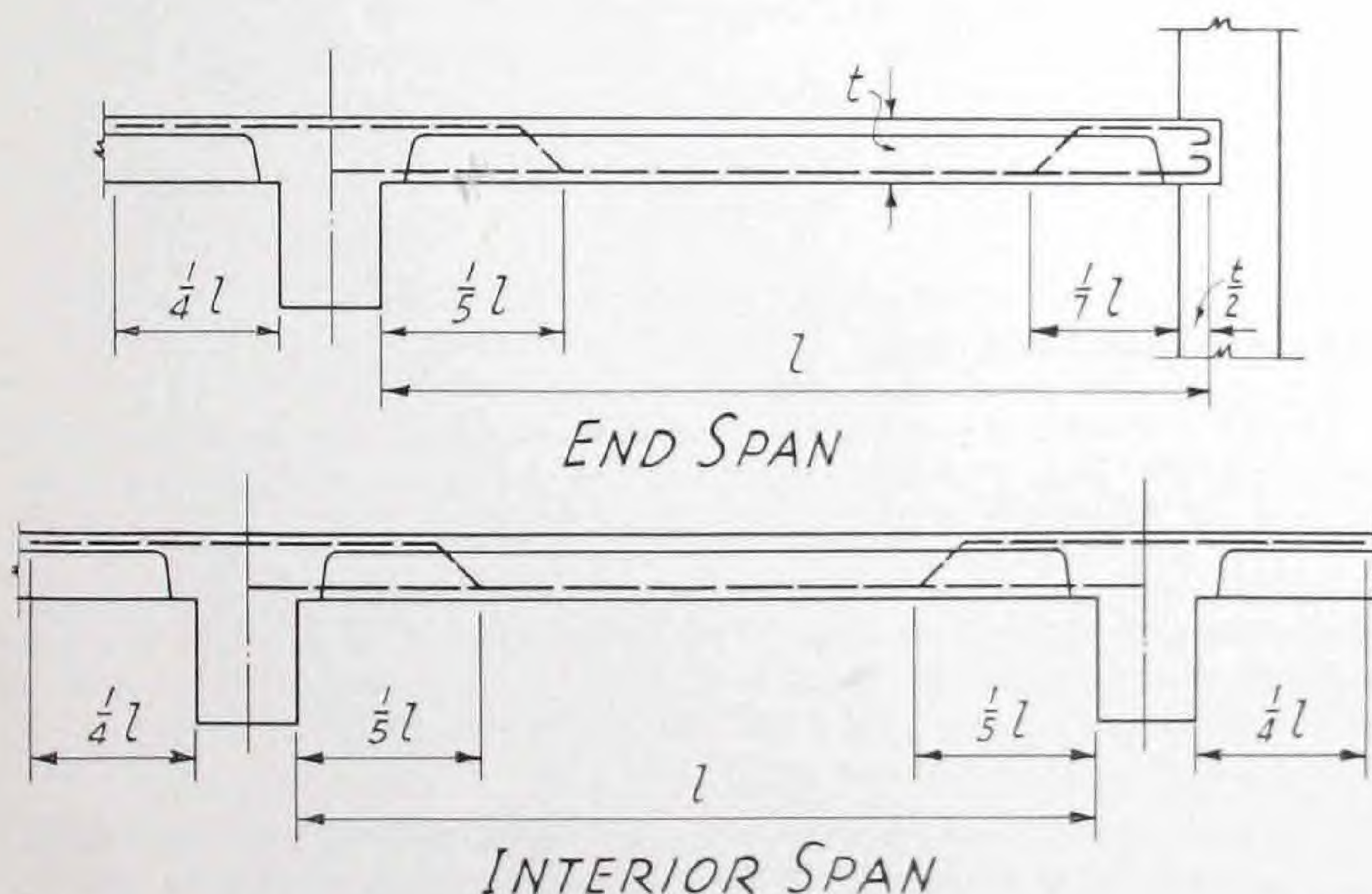


Fig. 13



## EXPLANATION OF TABLES No. 13 TO 16

Table No. 13 is a safe load table for beams of simple span designed for a moment of  $M = +\frac{1}{8}wl^2$  at the center of the span. Tables No. 14, 15 and 16 are for beams designed for moments at the support of  $-\frac{1}{8}wl^2$ ,  $-\frac{1}{10}wl^2$  and  $-\frac{1}{12}wl^2$ , respectively, and differ from the preceding tables in that the unit stress in the concrete,  $f_c$ , is taken as 900 p.s.i., instead of 800 p.s.i., as provided in the A. C. I. Code for compression in concrete at beam supports. Coefficients of  $wl^2$  for various span arrangements are given in Table 1, page 14. Typical details for bending bars are given in Fig. 14.

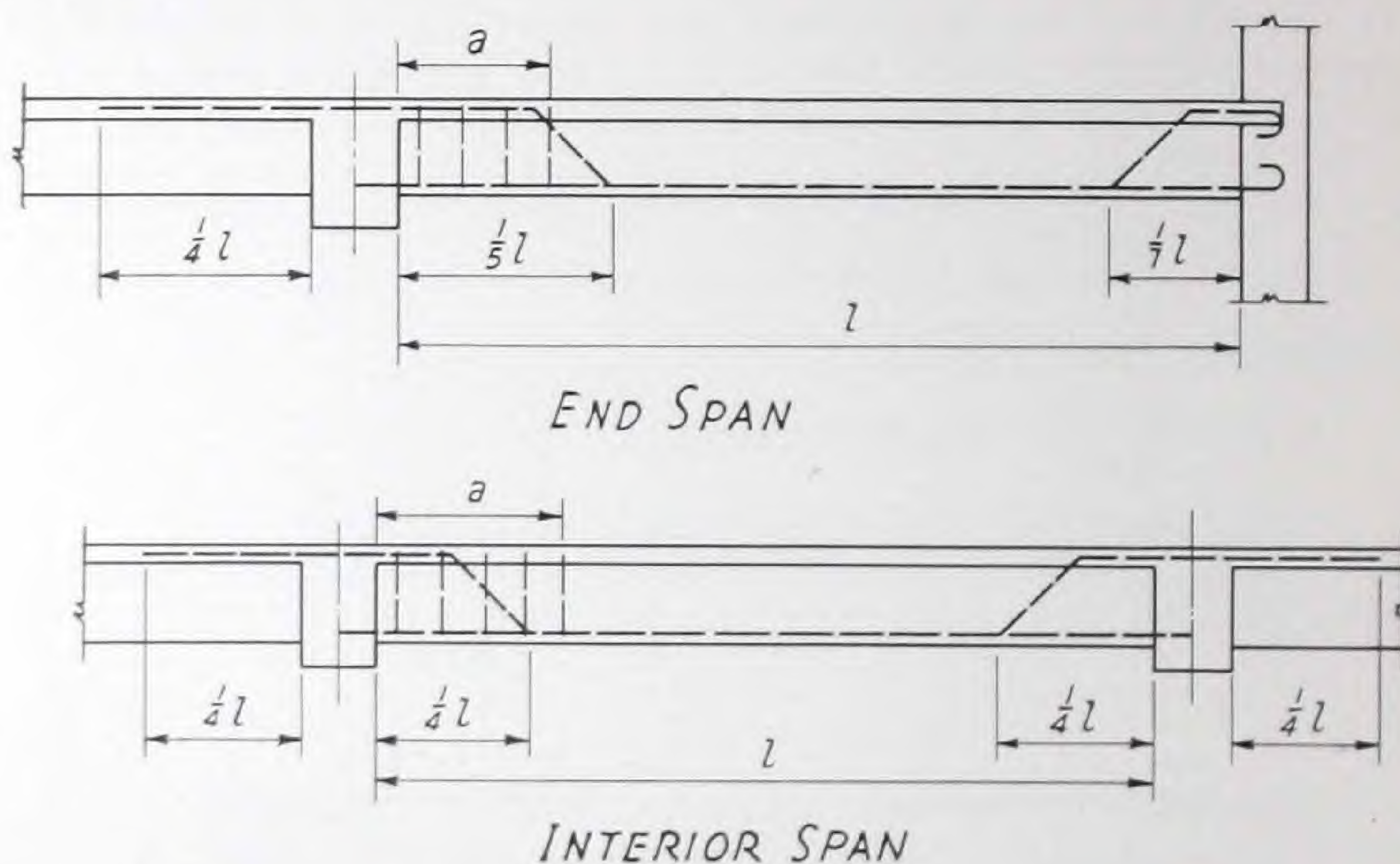


Fig. 14

### Illustrative Problem

Design a simple beam with a clear span of 22 ft. The beam carries a superimposed load, excluding the weight of the beam, of 1200 lb. per lin. ft. and the depth of the slab (thickness of flange) is 4 in.

Enter Table No. 13 and follow down the column for "span — 22 ft." until a safe superimposed load greater than 1200 p.l.f. is found opposite a flange thickness of 4 in., namely 1230 p.l.f. The depth of beam will be 14 in. given in Column 1 and the width of web will be 12 in., shown in Column 3. The weight of the web is 125 p.l.f.

The area of steel required for a superimposed load of 1230 lb. per lin. ft. is 4.68 sq. in. as found in Column 7. This may be reduced in proportion to the actual total load on the beam.



$$A_s = 4.68 \times \frac{1200 + 125}{1230 + 125} = 4.58 \text{ sq. in.}$$

According to Table No. 5, this area can be obtained with three  $1\frac{1}{4}$ -in. sq. bars. ( $A_s = 4.68$  sq. in.)

If the span had been 20 ft. the dimensions of the beam would be governed by shear and the area of steel would be determined by the design coefficients. The same dimensions would be required as in the previous case, the safe superimposed load being 1390 lb. per lin. ft. The steel coefficient,  $c_1$ , is 0.216 and  $A_s = c_1 l = 0.216 \times 20 = 4.32$  sq. in. This area can be obtained with three 1-in. rd. bars and two 1-in. sq. bars.  $A_s = 4.37$ .

The bond stress must be checked and shear reinforcement provided if necessary to complete the beam design. See Problems 2 to 6 for the method to be used.

## EXPLANATION OF TABLES No. 17 TO 22

The use of spans equal to clear distance between beam webs is specified in some building codes; in other codes, longer slab spans may be specified. In either case, the designer enters Tables No. 17 to 22 with values of *Spans in Feet* computed according to the code that governs; the use of the table is the same in either case.

The two-way concrete slabs in Tables No. 17 to 22 are designed according to the A. C. I. Code. The reinforcement given is that which is required in slab zones with maximum reinforcement, the balance of the reinforcement being determined as follows:

(1) Positive reinforcement adjacent to a continuous edge only and for width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.

(2) At a non-continuous edge, negative reinforcement per unit width, in amount at least as great as one-half that required for maximum positive moment for the center one-half of the panel, shall be provided across the entire width of the exterior support.

(3) The spacing of reinforcement shall be at most  $2\frac{1}{2}$  times the slab thickness and the ratio of reinforcement shall be at least 0.003.

Reinforcement in the long direction of the slab has been assumed to be in the top layer; some codes call for this reinforcement to be in the bottom layer. Economically, there is little difference between the two schemes.

Explanatory remarks, in addition to those in Problem 6, will be presented in the illustrative problem which follows. An end panel will be designed and the slab reinforcement detailed. The distribution of slab load to beams will also be discussed.



### Illustrative Problem

Given the panel in Fig. 15: end span is in the short direction and interior span in the long direction, the distance between centerlines of beams being 18 ft. and 24 ft. as shown. Let the superimposed load be the same as in Problem 6: 82 p.s.f. Assume a beam width of 12 in. for interior short beams, 14 in. for interior long beam and 10 in. for exterior beam. Determine the slab thickness, number, size and length of bars required.

*Design.* The ratio of long to short clear span is

$$\frac{24.0 - (0.5 + 0.5)}{18.0 - (0.6 + 0.4)} = \frac{23.0}{17.0} = 1.35$$

For design of slab, use Table No. 19 (Ratio 1.4 to 1.0). In column headed  $23.8 \times 17$  and for *End-Interior* condition of continuity, a 6-in. slab capable of supporting a superimposed load of 161 p.s.f. is seen to be sufficient. The 6-in. slab weighs 75 p.s.f.

Reinforcement given in table is:

Short way:  $\frac{1}{2}$ -in. rd. bars at 4-in. o.c. (area per linear foot = 0.60).

Long way:  $\frac{1}{2}$ -in. rd. bars at 6-in. o.c. (area per linear foot = 0.40).

The 6-in. slab with this reinforcement can carry safely a total load of  $161 + 75 = 236$  p.s.f., but the actual load is only  $82 + 75 = 157$  p.s.f. It is not permissible under the Code requirements to use a slab thinner than 6 in. for the conditions given. The reinforcement, however, may be reduced in the proportion of  $\frac{157}{236} = 0.67$ , but it shall be not less than that specified in point (3), which gives for minimum bar areas:

Short way:  $0.003 \times 12 \times (6 - 1) = 0.18$  sq. in.

Long way:  $0.003 \times 12 \times (6 - 1\frac{1}{2}) = 0.17$  sq. in.

Reinforcement required for strength according to Table No. 19 with correction allowed as calculated above is:

Short way ( $0.67 \times 0.60 = 0.40$ ):  $\frac{1}{2}$ -in. rd. bars at 6-in. o.c.

Long way ( $0.67 \times 0.40 = 0.27$ ):  $\frac{1}{2}$ -in. rd. bars at 9-in. o.c.

This reinforcement is to be placed in center bands. In the outer bands, the extent of which will be discussed later under detailing, the reinforcement may be reduced according to (1) as follows:

Short way ( $0.75 \times 0.67 \times 0.60 = 0.30$ ):  $\frac{1}{2}$ -in. rd. bars at 8-in. o.c.

Long way ( $0.75 \times 0.67 \times 0.40 = 0.20$ ):  $\frac{1}{2}$ -in. rd. bars at 12-in. o.c.

All of the four spacings are smaller than the maximum allowed in (3):  $2\frac{1}{2} \times 6 = 15$  in.



Detailing

For the slab designed above, placing plan, bar schedule and bending details are shown in Fig. 15. According to (1), the width of all outer bands, *b* and *e*, is  $\frac{1}{4} \times 17.0 = 4.25$ , where 17.0 is the *shorter* panel dimension.

The bars in band *a* are  $\frac{1}{2}$ -in. rd. at 6-in. o.c., the band width  $23.0 - 2 \times 4.25 = 14.5$  ft., and the number of bars  $\frac{12 \times 14.5}{6} = 29$ , of which 15 are straight and 14 are bent. For two bands *b*, the number of bars is  $\frac{12 \times 2 \times 4.25}{8} = 12$ , of which 6 are straight and 6 are bent.

For the outer band, *e*, in the long direction, the bars are  $\frac{1}{2}$ -in. rd. at 12-in. o.c., and the number of bars is  $\frac{12 \times 4.25}{12} = 4$ , of which 2 are straight and 2 are bent. The remainder of the short panel width,  $\frac{3}{4} \times 17.0$ , has  $\frac{1}{2}$ -in. rd. bars spaced 9-in. o.c., and the number of bars in this band, *d*, is  $\frac{12 \times 0.75 \times 17.0}{9} = 17$ , of which 9 are straight and 8 are bent.

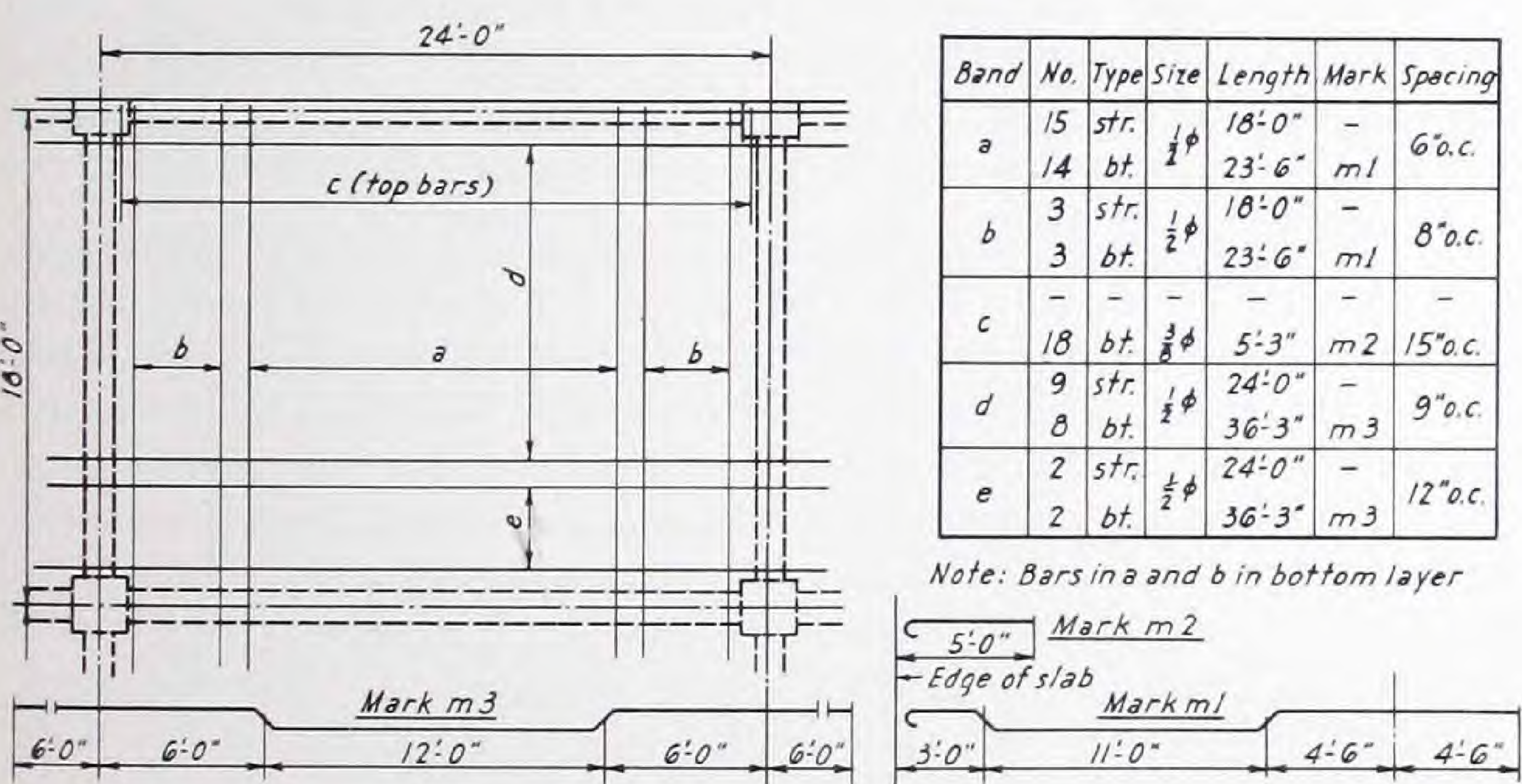


Fig. 15—PART FRAMING PLAN AND BAR SCHEDULE

Top bars in band *c* shall, according to code requirement, have an area per linear foot equal to one-half of that of bars in band *a*; that is, the reinforcement in *c* shall be  $\frac{1}{2}$ -in. rd. bars at 12-in. o.c., or equal. The bent bars in *b*,  $\frac{1}{2}$ -in. rd. at 16-in. are deficient in area, and all bent bars are bent up too close to the exterior beam to be fully effective as top bars. The deficiency will be made up by using additional top bars  $\frac{3}{8}$ -in. rd. at 15-in. o.c., the total number in *c* being  $\frac{12 \times 23}{15} = 18$  bars, which are hooked.



## ***Distribution of Load from Two-Way Slabs to Beams***

The load transmitted from a two-way solid slab to the supporting beams according to the A. C. I. Code, may be determined as follows: Let

$R$  = Slab reaction on beam in lb. per lin. ft.

$w$  = Total slab load in p.s.f.

$L$  = Longer span in ft.

$l$  = Shorter span in ft.

The reaction on the beams is then:

<i>For bending</i>	<i>For shear</i>
On longer beam: $R_L = (1 - er) \times \frac{wl}{2}$	and $(1 - r) \times \frac{wl}{2}$
On shorter beam: $R_l = (1 - er) \times \frac{wL}{2}$	and $(1 - r) \times \frac{wL}{2}$

The quantities  $(1 - er)$  and  $(1 - r)$  are used to denote a coefficient which is a function of  $L/l$  and the condition of continuity. The meaning and application of these quantities will be explained and illustrated in connection with Fig. 16, which refers to two-way slabs with ratio of  $L : l :: 1.4 : 1.0$ .

Fig. 16 shows nine groups with from one to nine slabs in each group marked from (1) to (9). Note that group 3-3 applies to floors having three or more spans in either direction. The numbers in parentheses represent all the types of "Condition of Continuity" described in the fourth and fifth columns of the two-way slab tables. Refer, for example, to Table No. 19, in which the condition of continuity in the first line for each slab thickness is: "Short Way Simple" and "Long Way Simple." Considering the factors applying to bending in beams, the first line in the box at the side of Table No. 19 gives—for the condition of "Simple-Simple"—the two values of  $(1 - er) : 0.835$  (for long beam) and  $0.440$  (for short beam). The condition of continuity: Simple-Simple, and the numbers:  $0.835$  and  $0.440$  are shown in slab (1) adjacent to the beams to which they apply. Similarly, each of the nine lines of two numbers in the boxes and the corresponding description of the continuity in the fourth and fifth columns given in the same horizontal line are shown in Fig. 16. If necessary, similar charts may readily be sketched for ratios other than  $1.4 : 1.0$ , and the values of  $(1 - er)$  and  $(1 - r)$  transferred from table to chart-form.

To illustrate the use of Fig. 16, consider again Fig. 15 and assume that the slab shown is part of a framing plan such as 3-3 in Fig. 16, the slab in Fig. 15 being the slab marked (6) in Fig. 16. The distance between center-lines of beams in slabs (5), (6) and (9) is 18 ft. the short way and 24 ft.



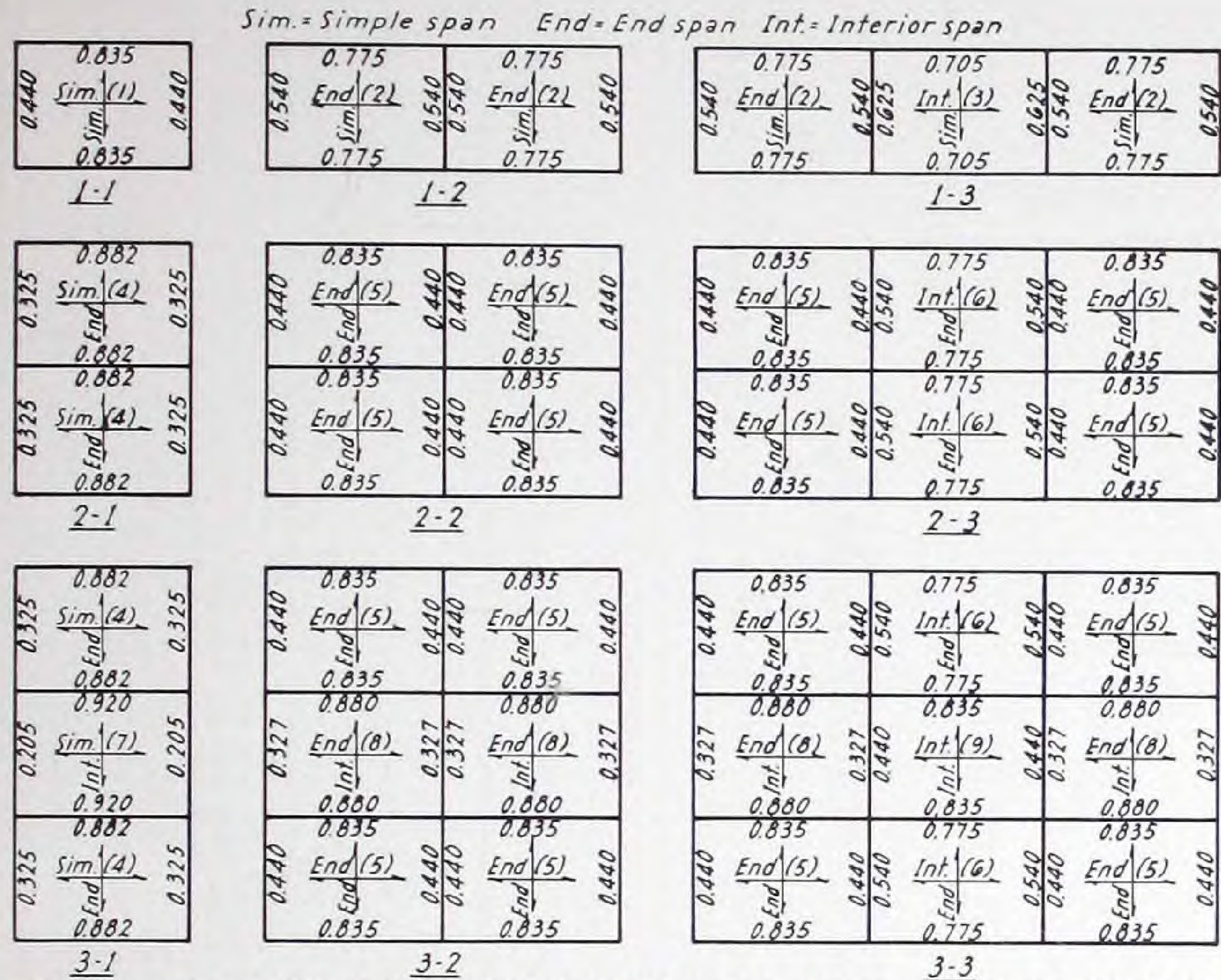


Fig. 16—FACTORS FOR DISTRIBUTION OF SLAB LOAD TO BEAMS FOR RATIO OF LONG TO SHORT SPAN = 1.4:1.0 FOR CALCULATION OF BEAM MOMENTS. SEE TABLE 19 FOR SIMILAR FACTORS FOR DETERMINATION OF BEAM SHEARS.

the long way. Determine equivalent loads to be used in calculating moments in beams along the four sides of slab (6), the total load being

$$82 + 69 = 151 \text{ p.s.f. on slab (9)}$$
$$82 + 75 = 157 \text{ p.s.f. on slab (6)}$$
$$82 + 81 = 163 \text{ p.s.f. on slab (5)}$$

The reaction on the beams in lb. per lin. ft. is as follows, exclusive of the weight of beam webs and using the formula given above with the value of  $(1 - er)$  taken from slabs (5), (6) and (9) in Fig. 16 or from lines 5, 6 and 9 in the box in Table No. 19:

Spandrel Beam:

$$0.775 \times 157 \times \frac{18}{2}$$

= 1100

Long Int. Beam:

$$0.775 \times 157 \times \frac{18}{2} + 0.835 \times 151 \times \frac{18}{2}$$

= 2230

Short Int. Beams:

$$0.540 \times 157 \times \frac{24}{2} + 0.440 \times 163 \times \frac{24}{2}$$

= 1880

These loads plus the weight of beam webs are to be used for calculating beam moments. For beam shears, use the same procedure but substitute values of  $(1 - r)$  for  $(1 - er)$ .



TABLE No. 7 — ONE-WAY SOLID CONCRETE SLABS

$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
 $\frac{3}{4}$ " clear fireproofing

SAFE SUPERIMPOSED LOAD IN LB. PER SQ. FT.																																				
SPANS IN FEET																																				
Total Thickness of Slab	Weight of Slab per Sq. Ft.	Volume of Concrete per Sq. Ft.	Re-quired Steel Area	Reinforcing Steel		Condi- tion of Con- tinuity of Span																														
				Size	Spacing																															
In.	Lb.	Cu. Ft.	Sq. In.	In.	In.	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12	12½	13	13½	14	14½	15	15½	16	16½	17	17½	18	
3	38	0.250	0.19		7	326	240	182	140	109	86	67	53	41																						
						309	236	184	146	116	94	75	61	49	39																					
							292	229	183	146	120	98	81	66	54	44	36																			
3½								296	231	183	147	119	96	78	63	51	41																			
	44	0.292	0.23		5½				300	240	195	159	131	109	90	75	62	51	42	31	49	34														
									297	242	200	167	139	117	99	83	70	59	48	39																
4	50	0.333	0.28		4½					275	223	182	150	125	103	86	71	59	48	39																
											292	241	201	168	142	120	102	86	73	56	47	39	32													
												299	251	212	181	154	132	114	98	84	72	61	52	44	37											
4½	56	0.375	0.32		7½						301	249		206	172	145	122	103	87	73	61	50	41	33												
													272	230	196	167	143	122	105	84	71	60	51	42	35											
														288	246	211	182	158	137	119	104	90	78	67	58											
5	63	0.417	0.36		6½								280	235	199	170	144	123	105	89	76	64	54	44	36											
															265	227	196	169	147	119	103	89	77	65	56											
																286	249	216	189	165	145	127	112	98	86											
5½	69	0.458	0.41		5½										264	226	194	167	144	124	107	92	79	67	57											
																	259	226	197	162	141	123	108	93	81											
																		250	220	195	172	152	135	120												
6	75	0.500	0.45		5																															







$f_s=20,000$  p.s.i.

$f_c=800$  p.s.i.

$n=15$

Max.  $v=40$  p.s.i.

$\frac{3}{4}$ " clear fireproofing

$$A_s = \frac{\text{Total Load per sq. ft.} \times \text{Steel Coeff. (sq. in. per joist)}}{\text{Load Coeff.}}$$

Tiles to be staggered.

( $1\frac{1}{2}$ " Added to effective width of joist.)

TABLE No. 8—CONCRETE OR CLAY TILE  
AND CONCRETE JOISTS

SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																												
Slab (Tile + Topping) Total Depth	Effective Depth In.	Width of Joist In.	Weight per Sq. Ft. Slab Lb.	Volume of Concrete Cu. Ft. per Sq. Ft.	STEEL COEFFICIENTS FOR MOMENTS OF			SPANS IN FEET																				
					$\frac{wl^2}{8}$	$\frac{wl^2}{10}$	$\frac{wl^2}{12}$	Load	Coefficient→	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28
4+2	4.88	4 5 6	49 51 52	0.250 0.265 0.278	374	309	260	221	191	166	146	129																
					91	79	68	59	51	45	39	41																
					106	90	79	69	61	53	47	48																
4+2½	5.38	4 5 6	55 57 58	0.292 0.306 0.319	455	376	316	269	232	202	178	157	140															
					100	86	74	64	56	48	42	36																
					116	100	87	76	66	58	51	45	39															
4+3	5.88	4 5 6	62 64 65	0.333 0.348 0.361	544	449	378	322	277	242	212	188	168															
					108	92	79	68	59	51	44	38																
					125	108	93	81	71	62	54	47	41															
6+2	6.88	4 5 6	60 62 65	0.292 0.314 0.333	715	591	497	423	365	318	280	248	221	198	179	162	148											
					141	123	107	95	83	74	65	58	51	46	40	44	40											
					162	142	124	110	98	87	78	70	62	56	50	51	45											
6+2½	7.31	4 5 6	66 68 71	0.333 0.356 0.375	835	690	580	494	426	371	326	289	258	231	208	189	172											
					145	126	110	96	85	75	66	58	51	45	39	44	39											
					167	146	128	113	100	89	79	70	62	56	49	51	45											
					185	162	143	126	112	100	89	80	71	64	57	51	45	198	218	240	266	296	332	375	427	491		
					961	794	667	569	491	427	375	332	296	266	240	218	198											







$f_g = 20,000$  p.s.i.,  
 $f_c = 800$  p.s.i.,  
 $n = 15$   
 Max.  $v = 40$  p.s.i.,  
 $\frac{3}{4}$ " clear fireproofing

TABLE No. 9—20" METAL PANS AND  
 CONCRETE JOISTS (STRAIGHT PANS)

$$A_s = \frac{\text{Total Load per sq. ft.} \times \text{Steel Coeff. (sq. in. per joist)}}{\text{Load Coeff.}}$$

SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																											
SPANS IN FEET																											
Slab (Pan + Topping) Total Depth	Effective Depth	Width of Joist	Weight per Sq. Ft. Slab	Volume of Concrete Cu. Ft. per Sq. Ft.	STEEL COEFFICIENTS FOR MOMENTS OF			10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	
					$\frac{wl^2}{8}$	$\frac{wl^2}{10}$	$\frac{wl^2}{12}$																				
					Load	Coefficient→																					
6+2	6.81	4 5 6	39 42 44	0.26 0.28 0.29	Load	705	582	490	417	360	313	275															
					1.75	1.40	1.17	57	48	41																	
					1.83	1.46	1.22	73	63	54	46	40															
6+2½	7.31	4 5 6	46 48 50	0.30 0.32 0.33	Load	836	691	581	495	427	371	327															
					1.95	1.56	1.30	56	47	39																	
					2.04	1.63	1.36	75	63	54	46	39															
8+2	8.75	4 5 6	45 48 51	0.30 0.32 0.34	Load	1053	871	732	624	538	468	412	365	325	298												
					2.00	1.60	1.34	81	70	60	52	45															
					2.09	1.67	1.39	103	90	78	68	60	53	46	41	46	41										
8+2¾	9.25	4 5 6	51 54 57	0.38	Load	1264	1045	878	748	645	562	494	438	390													
					2.30	1.84	1.53	81	68	58	50	43															
					2.40	1.92	1.60	104	89	77	67	59	51	45	50	44											
10+2	10.75	4	51	0.34	Load	1437	1178	990	845	728	634	557	493	440	395	366	323	295									
					2.17	1.73	1.45	107	92	80	70	62	54	47	42												







$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
Max.  $v = 40$  p.s.i.  
 $\frac{3}{4}$ " clear fireproofing

TABLE No: 10 — 20" METAL PANS AND  
CONCRETE JOISTS (TAPERED PANS)

$$A_s = \frac{\text{Total Load per sq. ft.}}{\text{Load Coeff.}} \times \text{Steel Coeff. (sq. in. per joist)}$$

Slab (Pan + Topping) Total Depth	Effective Depth	Width of Joist	Approx. Wt. Per Sq. Ft. of Slab	STEEL COEFFICIENTS FOR MOMENTS OF			SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																	
				$\frac{wl^3}{8}$	$\frac{wl^2}{10}$	$\frac{wl^2}{12}$	SPANS IN FEET																	
In.	In.	In.	Lb.	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28		
6+2	6.81	4 5 6	43 46 48	Load	582	490	417	360	313	275	244	218	195	176	160	146	133	122	113					
				1.75	1.40	1.17	95	85	77	70	64	58	53	49	45	41								
				1.83	1.46	1.22	103	93	84	76	69	63	58	53	48	44	41							
6+2½	7.31	4 5 6	50 52 54	Load	691	581	495	427	371	327	289	258	232	209	189	173	158	145	134					
				1.95	1.56	1.30	96	87	78	71	64	58	52	48	43	44	40							
				2.04	1.63	1.36	106	95	86	78	71	64	58	53	48	44	40	41						
8+2	8.75	4 5 6	50 53 56	Load	871	732	624	538	468	412	365	325	298	264	239	218	199	183	169	156	145	134		
				2.00	1.60	1.34	131	119	108	99	91	83	76	70	65	60	55	51	47	44	40			
				2.09	1.67	1.39	142	129	118	108	99	91	83	77	71	66	61	56	52	48	45			
8+2½	9.25	4 5 6	56 59 62	Load	1045	878	748	645	562	494	438	390	350	316	287	262	239	220	202	187	174	161		
				2.30	1.84	1.53	132	119	108	99	90	82	75	69	63	58	54	49	45	41	43			
				2.40	1.92	1.60	144	131	119	108	99	91	83	76	70	65	59	55	50	46	43			
10+2	10.75	4	56	Load		990	845	728	634	557	493	440	395	366	323	295	270	248	228	211	196	182		
				2.17	1.73	1.45	170	154	142	130	119	110	102	94	88	81	76	70	65	61	57			







TABLE No. 11 — 30" METAL  
PANS AND CONCRETE  
JOISTS (STRAIGHT PANS)

$$A_s = \frac{\text{Total Load per sq. ft.}}{\text{Load Coeff.}} \times \text{Steel Coeff. (sq. in. per joist)}$$

$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
Max.  $v = 40$  p.s.i.  
 $\frac{3}{4}$ " clear fireproofing

SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																										
					SPANS IN FEET																					
					STEEL COEFFICIENTS FOR MOMENTS OF																					
					$\frac{wl^3}{8}$	$\frac{wl^2}{10}$	$\frac{wl^3}{12}$	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25			
Slab (Pan + Topping) Total Depth	Effective Depth	Width of Joist	Weight per Sq. Ft. Slab	Volume of Concrete Cu. Ft. per Sq. Ft.	Load	Coefficient→	824	681																		
					2.75	2.20	1.84	31	36																	
					2.83	2.27	1.89	44	47																	
					2.92	2.33	1.94	56																		
6+2½	7.25	4 5 6	41 43 45	0.27 0.29 0.30	Load	Coefficient→	1264	1044	878	749	645	562														
					3.26	2.61	2.17	48	39																	
					3.35	2.68	2.24	65	55																	
					3.45	2.76	2.30	82	69																	
8+2½	9.25	4 5 6	45 48 50	0.30 0.32 0.33	Load	Coefficient→	1457	1204	1010	862	743	647														
					3.60	2.88	2.40	46	37																	
					3.71	2.97	2.47	64	53																	
					3.81	3.05	2.54	81	69																	
8+3	9.75	4 5 6	51 54 56	0.34 0.36 0.37	Load	Coefficient→	1720	1420	1194	1017	878	765	672	595	531											
					3.60	2.88	2.40	66	55																	
					3.71	2.97	2.47	81	69																	
					3.81	3.05	2.54	98	81																	
10+2½	11.25	4	49	0.33	Load	Coefficient→	2016	1680	1411	1216	1044	900	781	688	611	549	499	457	422	392	366	343	322			
					3.60	2.88	2.40	88	73																	
					3.71	2.97	2.47	104	87																	
					3.81	3.05	2.54	121	101																	







$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
Max.  $v = 40$  p.s.i.  
 $\frac{3}{4}$ " clear fireproofing

TABLE No. 12—30" METAL PANS AND  
CONCRETE JOISTS (TAPERED PANS)

$$A_s = \frac{\text{Total Load per sq. ft.} \times \text{Steel Coeff. (sq. in. per joist)}}{\text{Load Coeff.}}$$

SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																										
Slab (Pan + Topping) Total Depth	Effective Depth	Width of Joist	Approx. Wt. Per Sq. Ft. of Slab	STEEL COEFFICIENTS FOR MOMENTS OF			SPANS IN FEET																			
				$\frac{wl^2}{8}$	$\frac{wl^2}{10}$	$\frac{wl^2}{12}$	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	
				Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient	→	Load	Coefficient
In.	In.	In.	Lb.																							
6+2½	7.25	4 5 6	44 46 48	824	681	572	488	420	366	322	285	254	228													
				99	86	76	66	58	52	46	40															
				111	96	85	75	66	59	52	46	41														
8+2½	9.25	4 5 6	49 52 54	1264	1044	878	749	645	562	494	438	390	350	316	287	262										
				137	120	106	94	84	75	67	60	49	44													
				151	132	117	104	93	83	75	67	61	55	49	45	40										
8+3	9.75	4 5 6	55 58 60	1457	1204	1010	862	743	647	569	504	450	404	364	330	301	275									
				139	121	106	94	83	74	66	59	47	42													
				154	135	119	105	93	83	74	67	60	54	48	43	40										
10+2½	11.25	4	53	1720	1420	1194	1017	878	765	672	595	531	476	430	390	356	325	298	275	254	236					
				176	155	138	123	111	100	90	82	74	68	61	56	51	47	43								
				3.60	2.88	2.40																				



10+3	11.75	4 5 6	60 63 65	Load Coefficient→										1998	1650	1388	1182	1020	888	781	692	616	554	499	453	413	378	347	320	296																										
				4.04	3.23	2.69	177	156	137	122	109	98	88																				79	72	65	58	53	48	43																	
				4.16	3.33	2.77	196	173	153	136	122	110	99																				89	81	73	66	60	55	50	45	41															
				4.28	3.43	2.86	215	189	169	150	135	122	110																				100	91	82	75	68	62	57	52	47	43														
12+2½	13.25	5 6 7 8	61 65 68 71	Load Coefficient→													1293	1115	972	855	756	675	605	547	495	452	414	379	350	324	300	279																								
				3.94	3.15	2.63			188	169	153	138	126																						115	105	96	88	81	75	69	63	59	54	50	46										
				4.05	3.24	2.70			205	184	167	141	137																						126	115	106	97	89	82	76	70	65	59	55	51										
				4.16	3.33	2.78			221	198	180	163	148																						136	125	114	105	97	90	83	76	71	65	60	56										
12+3	13.75	5 6 7 8	67 71 74 77	Load Coefficient→												1767	1504	1298	1130	994	880	785	705	636	576	526	481	441	407	376	349	325																								
				4.47	3.58	2.98			189	169	152	138	125																						114	103	95	86	79	73	66	61	56	51	47	43										
				4.60	3.68	3.07			206	184	166	151	137																						124	114	104	95	87	80	73	67	62	57	52	47	43									
				4.73	3.78	3.15			222	200	180	163	148																						135	123	113	104	95	88	81	74	68	63	58	53										
14+2½	15.25	5 6 7 8	66 70 74 78	Load Coefficient→														1351	1176	1034	916	817	734	662	600	546	500	460	424	392	363	338																								
				4.10	3.28	2.73					183	166	152																						139	127	117	108	100	92	85	79	73	68	63	58										
				4.21	3.37	2.81					198	181	165																						151	139	128	118	109	101	93	87	80	74	69	64										
				4.33	3.46	2.89					214	194	178																						163	150	138	127	118	109	101	94	87	81	75	70										
14+3	15.75	5 6 7 8	73 76 80 84	Load Coefficient→														1580	1378	1210	1072	957	859	775	703	640	586	538	496	458	425	396																								
				4.70	3.76	3.14					181	164	149																						136	124	114	104	96	88	81	75	69	64	58	54										
				4.84	3.87	3.22					198	180	164																						150	138	126	116	107	99	91	84	78	72	66	61										
				4.97	3.98	3.31					214	194	177																						162	148	136	125	116	107	99	91	84	78	72	66	61									
			84	Load Coefficient→														228	208	189	173	159	146	135	124	115	106	98	91	84	78	72																								
				5.10	4.08	3.40																																																		



$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
 $1\frac{1}{2}$ " clear fireproofing

TABLE No. 13 — CONCRETE T-BEAMS

SIMPLE SPANS  
 $M = +\frac{1}{8}wl^2$

Total Depth of Beam In.	Effective Depth In.	Width of Web In.	Thickness of Flange In.	Weight of Web per Ft. Beam Lb.	Width of Flange In.	Area of Steel Sq. In.	Reinforcing Steel		Design Coefficients		SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
							Straight In.	Bent In.	Reinf. Steel $c_1$ ( $A_s=c_1l$ )	Flange Width $c_2$ ( $b=c_2l$ )	SPANS IN FEET ( $l$ )																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
12	10	8 10	4 4	67 83	30 34	2.27 2.54	1-1 $\frac{1}{8}$ " 1-1 $\frac{1}{8}$ "	1-1 $\frac{1}{8}$ " 1-1 $\frac{1}{8}$ "	0.144 0.180	1.92 2.40	1330 1670	1230 1530	1130 1420	1050 1240	970 1080	850 950	750 830	670 740	600 660	530 590	480 530																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										



18	16	14	12	10	8	6	5	4	3	2	1	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100	101	102	103	104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192	193	194	195	196	197	198	199	200	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220	221	222	223	224	225	226	227	228	229	230	231	232	233	234	235	236	237	238	239	240	241	242	243	244	245	246	247	248	249	250	251	252	253	254	255	256	257	258	259	260	261	262	263	264	265	266	267	268	269	270	271	272	273	274	275	276	277	278	279	280	281	282	283	284	285	286	287	288	289	290	291	292	293	294	295	296	297	298	299	300	301	302	303	304	305	306	307	308	309	310	311	312	313	314	315	316	317	318	319	320	321	322	323	324	325	326	327	328	329	330	331	332	333	334	335	336	337	338	339	340	341	342	343	344	345	346	347	348	349	350	351	352	353	354	355	356	357	358	359	360	361	362	363	364	365	366	367	368	369	370	371	372	373	374	375	376	377	378	379	380	381	382	383	384	385	386	387	388	389	390	391	392	393	394	395	396	397	398	399	400	401	402	403	404	405	406	407	408	409	410	411	412	413	414	415	416	417	418	419	420	421	422	423	424	425	426	427	428	429	430	431	432	433	434	435	436	437	438	439	440	441	442	443	444	445	446	447	448	449	450	451	452	453	454	455	456	457	458	459	460	461	462	463	464	465	466	467	468	469	470	471	472	473	474	475	476	477	478	479	480	481	482	483	484	485	486	487	488	489	490	491	492	493	494	495	496	497	498	499	500	501	502	503	504	505	506	507	508	509	510	511	512	513	514	515	516	517	518	519	520	521	522	523	524	525	526	527	528	529	530	531	532	533	534	535	536	537	538	539	540	541	542	543	544	545	546	547	548	549	550	551	552	553	554	555	556	557	558	559	560	561	562	563	564	565	566	567	568	569	570	571	572	573	574	575	576	577	578	579	580	581	582	583	584	585	586	587	588	589	590	591	592	593	594	595	596	597	598	599	600	601	602	603	604	605	606	607	608	609	610	611	612	613	614	615	616	617	618	619	620	621	622	623	624	625	626	627	628	629	630	631	632	633	634	635	636	637	638	639	640	641	642	643	644	645	646	647	648	649	650	651	652	653	654	655	656	657	658	659	660	661	662	663	664	665	666	667	668	669	670	671	672	673	674	675	676	677	678	679	680	681	682	683	684	685	686	687	688	689	690	691	692	693	694	695	696	697	698	699	700	701	702	703	704	705	706	707	708	709	710	711	712	713	714	715	716	717	718	719	720	721	722	723	724	725	726	727	728	729	730	731	732	733	734	735	736	737	738	739	740	741	742	743	744	745	746	747	748	749	750	751	752	753	754	755	756	757	758	759	760	761	762	763	764	765	766	767	768	769	770	771	772	773	774	775	776	777	778	779	780	781	782	783	784	785	786	787	788	789	790	791	792	793	794	795	796	797	798	799	800	801	802	803	804	805	806	807	808	809	810	811	812	813	814	815	816	817	818	819	820	821	822	823	824	825	826	827	828	829	830	831	832	833	834	835	836	837	838	839	840	841	842	843	844	845	846	847	848	849	850	851	852	853	854	855	856	857	858	859	860	861	862	863	864	865	866	867	868	869	870	871	872	873	874	875	876	877	878	879	880	881	882	883	884	885	886	887	888	889	890	891	892	893	894	895	896	897	898	899	900	901	902	903	904	905	906	907	908	909	910	911	912	913	914	915	916	917	918	919	920	921	922	923	924	925	926	927	928	929	930	931	932	933	934	935	936	937	938	939	940	941	942	943	944	945	946	947	948	949	950	951	952	953	954	955	956	957	958	959	960	961	962	963	964	965	966	967	968	969	970	971	972	973	974	975	976	977	978	979	980	981	982	983	984	985	986	987	988	989	990	991	992	993	994	995	996	997	998	999	1000	1001	1002	1003	1004	1005	1006	1007	1008	1009	1010	1011	1012	1013	1014	1015	1016	1017	1018	1019	1020	1021	1022	1023	1024	1025	1026	1027	1028	1029	1030	1031	1032	1033	1034	1035	1036	1037	1038	1039	1040	1041	1042	1043	1044	1045	1046	1047	1048	1049	1050	1051	1052	1053	1054	1055	1056	1057	1058	1059	1060	1061	1062	1063	1064	1065	1066	1067	1068	1069	1070	1071	1072	1073	1074	1075	1076	1077	1078	1079	1080	1081	1082	1083	1084	1085	1086	1087	1088	1089	1090	1091	1092	1093	1094	1095	1096	1097	1098	1099	1100	1101	1102	1103	1104	1105	1106	1107	1108	1109	1110	1111	1112	1113	1114	1115	1116	1117	1118	1119	1120	1121	1122	1123	1124	1125	1126	1127	1128	1129	1130	1131	1132	1133	1134	1135	1136	1137	1138	1139	1140	1141	1142	1143	1144	1145	1146	1147	1148	1149	1150	1151	1152	1153	1154	1155	1156	1157	1158	1159	1160	1161	1162	1163	1164	1165	1166	1167	1168	1169	1170	1171	1172	1173	1174	1175	1176	1177	1178	1179	1180	1181	1182	1183	1184	1185	1186	1187	1188	1189	1190	1191	1192	1193	1194	1195	1196	1197	1198	1199	1200	1201	1202	1203	1204	1205	1206	1207	1208	1209	1210	1211	1212	1213	1214	1215	1216	1217	1218	1219	1220	1221	1222	1223	1224	1225	1226	1227	1228	1229	1230	1231	1232	1233	1234	1235	1236	1237	1238	1239	1240	1241	1242	1243	1244	1245	1246	1247	1248	1249	1250	1251	1252	1253	1254	1255	1256	1257	1258	1259	1260	1261	1262	1263	1264	1265	1266	1267	1268	1269	1270	1271	1272	1273	1274	1275	1276	1277	1278	1279	1280	1281	1282	1283	1284	1285	1286	1287	1288	1289	1290	1291	1292	1293	1294	1295	1296	1297	1298	1299	1300	1301	1302	1303	1304	1305	1306	1307	1308	1309	1310	1311	1312	1313	1314	1315	1316	1317	1318	1319	1320	1321	1322	1323	1324	1325	1326	1327	1328	1329	1330	1331	1332	1333	1334	1335	1336	1337	1338	1339	1340	1341	1342	1343	1344	1345	1346	1347	1348	1349	1350	1351	1352	1353	1354	1355	1356	1357	1358	1359	1360	1361	1362	1363	1364	1365	1366	1367	1368	1369	1370	1371	1372	1373	1374	1375	1376	1377	1378	1379	1380	1381	1382	1383	1384	1385	1386	1387	1388	1389	1390	1391	1392	1393	1394	1395	1396	1397	1398	1399	1400	1401	1402	1403	1404	1405	1406	1407	1408	1409	1410	1411	1412	1413	1414	1415	1416	1417	1418	1419	1420	1421	1422	1423	1424	1425	1426	1427	1428	1429	1430	1431	1432	1433	1434	1435	1436	1437	1438	1439	1440	1441	1442	1443	1444	1445	1446	1447	1448	1449	1450	1451	1452	1453	1454	1455	1456	1457	1458	1459	1460	1461	1462	1463	1464	1465	1466	1467	1468	1469	1470	1471	1472	1473	1474	1475	1476	1477	1478	1479	1480	1481	1482	1483	1484	1485	1486	14
----	----	----	----	----	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	------	----



TABLE No. 13 Continued—CONCRETE T-BEAMS

$f_s = 20,000$  p.s.i.  $n = 15$   
 $f_c = 800$  p.s.i.  $1\frac{1}{2}''$  clear fireproofing

SIMPLE SPANS  
 $M = +\frac{1}{8}wl^2$

Total Effective Depth of Beam		Width of Web		Thickness of Flange		Weight of Web per Ft. of Beam		Width of Flange		Area of Steel		Reinforcing Steel (2 Layers) in Inches		Design Coefficients		SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																				
																SPANS IN FEET (')																				
In.	In.	In.	In.	In.	In.	Lb.	In.	In.	Sq. In.	Sq. In.	Sq. In.	Sq. In.	Sq. In.	Reinf. Steel $c_1$ ( $A_s = c_1 l$ )	Flange Width $c_2$ ( $b = c_2 l$ )	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
22	18.88	8	35	4	150	35	4.03	2-1 □	2-1 1/8 □	2-1 □	0.144	1.252	2050	1910	1790	1680	1580	1500	1420	1350	1280	1220	1170	1120	1070	1020	940	870	800	740	690	640	600	560		
													1990	1860	1740	1640	1550	1460	1390	1320	1250	1190	1140	1090	1050	1000	920	850	790	730	680	640	590	550		
													2000	1860	1750	1640	1550	1470	1390	1320	1260	1200	1150	1100	1060	1010	930	870	800	750	690	650	600	560		
													2550	2380	2230	2100	1980	1870	1770	1680	1600	1530	1460	1390	1340	1280	1190	1100	1020	950	880	820	760	710		
		10	37	6	167	37	5.03	2-1 1/8 □	2-1 1/8 □	0.180	1.302	2490	2320	2170	2040	1930	1820	1730	1640	1560	1490	1420	1360	1310	1250	1170	1090	1010	930	870	810	750	700			
												2490	2330	2180	2050	1940	1830	1740	1650	1570	1500	1440	1380	1320	1270	1190	1100	1020	950	880	820	770	720			
												3070	2860	2630	2520	2380	2250	2130	2020	1930	1840	1750	1680	1610	1540	1480	1380	1280	1190	1100	1030	960	890			
												2990	2790	2510	2460	2320	2190	2080	1970	1880	1790	1710	1640	1570	1510	1450	1360	1260	1170	1090	1010	940	880			
		12	45	8	175	45	6.32	4-1 φ	4-1 φ	0.216	1.524	2990	2800	2620	2470	2330	2200	2090	1990	1890	1810	1730	1650	1590	1520	1460	1380	1280	1190	1110	1030	960	900			
												3580	3340	3130	2940	2770	2620	2480	2360	2240	2140	2040	1950	1870	1800	1680	1560	1440	1330	1240	1150	1070	1000			
												3480	3250	3050	2870	2700	2550	2420	2300	2190	2090	2000	1910	1830	1760	1660	1530	1420	1320	1230	1140	1060	990			
												3500	3270	3050	2880	2720	2570	2440	2320	2210	2110	2020	1930	1850	1780	1670	1550	1440	1340	1250	1160	1090	1020			
		14	62	4	263	62	7.16	4-1 □	4-1 □	0.252	2.195	4090	3810	3570	3360	3170	2990	2840	2690	2560	2440	2330	2230	2140	2050	1970	1890	1820	1700	1580	1470	1370	1280			
												3980	3720	3480	3270	3090	2920	2770	2630	2500	2390	2280	2190	2090	2010	1930	1860	1790	1670	1550	1450	1350	1260			
												3990	3730	3500	3290	3100	2940	2790	2650	2530	2410	2300	2210	2120	2030	1950	1880	1810	1690	1580	1480	1380	1290			
												2850	2660	2490	2340	2210	2090	1980	1880	1790	1700	1630	1560	1490	1430	1370	1320	1270	1190	1110	1030	960	900			
24	20.88	10	38	6	188	38	5.66	2-1 1/4 □	2-1 1/8 □	0.180	1.215	2770	2590	2420	2280	2150	2030	1930	1830	1740	1660	1590	1520	1460	1400	1340	1290	1240	1170	1090	1020	950	890			
												2750	2570	2410	2260	2140	2020	1920	1820	1740	1660	1580	1520	1450	1400	1340	1290	1250	1180	1100	1030	960	900			
												3420	3190	2990	2810	2650	2500	2370	2250	2140	2040	1950	1870	1790	1710	1650	1580	1520	1470	1420	1320	1230	1150			
												3330	3110	2910	2740	2580	2440	2320	2200	2090	2000	1910	1830	1750	1680	1610	1550	1500	1440	1390	1310	1220	1140			
		12	46	8	200	46	7.16	4-1 □	4-1 φ	0.216	1.380	3300	3080	2890	2720	2560	2430	2300	2190	2080	1990	1900	1820	1740	1670	1610	1550	1490	1440	1350	1260	1170				
												3990	3720	3490	3280	3090	2920	2770	2630	2500	2390	2280	2180	2090	2000	1930	1850	1780	1690	1570	1460	1360	1270			
												3890	3630	3400	3200	3010	2850	2700	2570	2450	2330	2230	2130	2040	1960	1890	1810	1750	1660	1550	1440	1350	1260			
												3860	3610	3380	3180	3000	2840	2690	2560	2440	2330	2220	2130	2040	1960	1880	1810	1750	1670	1560	1460	1360	1270			
		14	51	8	234	51	8.00	4-1 □	4-1 □	0.252	1.610	4570	4270	3990	3750	3540	3340	3170	3010	2860	2730	2610	2490	2390	2290	2200	2120	2040	1960	1900	1830	1650	1540			
												4440	4150	3890	3650	3450	3260	3090	2930	2790	2670	2550	2440	2340	2240	2150	2070	2000	1920	1860	1790	1710	1600			
												4410	4120	3860	3630	3420	3240	3070	2920	2780	2660	2540	2430	2330	2240	2150	2070	1990	1920	1860	1790	1720	1620			
												0.288	2.414	0.288	1.944	0.288	1.840	0.180	1.464	3140	2930	2740	2580	2430	2300	2180	2070	1970	1870	1790	1710	1630	1560	1490	1430	1360
25	22.88	10	68	6	208	68	10.00	5-1 □	5-1 □	0.288	1.840	4440	4150	3890	3650	3450	3260	3090	2930	2790	2670	2550	2440	2340	2240	2150	2070	2000	1920	1860	1790	1710	1600			
												4410	4120	3860	3630	3420	3240	3070	2920	2780	2660	2540	2430	2330	2240	2150	2070	1990	1920	1860	1790	1720	1620			
												0.288	1.840	0.180	1.464	3140	2930	2740	2580	2430	2300	2180	2070	1970	1870	1790	1710	1630	1560	1490	1430	1360	1290	1210	1170	
												0.180	1.152	3050	2850	2670	2510	2370	2240	2120	2010	1920	1830	1750	1670	1600	1540	1480	1420	1370	1320	1270	1230	1190	1150	
		12	60	4	250	60	7.16	4-1 □	4-1 φ	0.216	1.810	3420	3190	2990	2810	2650	2500	2370	2250	2140	2040	1950	1870	1790	1710	1650	1580	1520	1470	1420	1320	1230	1150			
												3330	3110	2910	2740	2580	2440	2320	2200	2090	2000	1910	1830	1750	1680	1610	1550	1500	1440	1390	1310	1220	1140			
												3990	3720	3490	3280	3090	2920	2770	2630																	



$f_s=20,000$  p.s.i.  
 $f_c=800$  p.s.i.  
 $n=15$   
1½" clear fireproofing

TABLE No. 13 Continued—CONCRETE T-BEAMS

SIMPLE SPANS  
 $M = +\frac{1}{8}wl^2$

SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																												
Total Depth of Beam In.	Effective Depth In.	Width of Web In.	Thickness of Flange In.	Weight of Web per Ft. of Beam Lb.	Design Coefficients		SPANS IN FEET (l)																					
					Reinf. Steel $c_1$ ( $A_s=c_1l$ )	Flange Width $c_2$ ( $b=c_2l$ )	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
28	24.75	10	6	229	0.180	1.105	3330	3100	2910	2730	2580	2440	2310	2190	2090	1990	1910	1820	1750	1670	1610	1550	1490	1440	1390	1340	1290	1250
				208	0.180	0.990	3270	3060	2860	2690	2540	2400	2280	2170	2060	1970	1880	1800	1730	1660	1590	1530	1480	1420	1370	1330	1280	1240
				188	0.180	0.970	3280	3060	2870	2700	2550	2410	2290	2180	2070	1980	1890	1810	1740	1670	1600	1550	1490	1440	1390	1340	1300	1260
	12	6	275	0.216	1.325	3990	3730	3490	3280	3100	2930	2780	2640	2510	2390	2290	2190	2100	2010	1930	1860	1790	1730	1670	1610	1550	1500	
			250	0.216	1.188	3930	3670	3430	3230	3050	2880	2730	2600	2470	2360	2250	2160	2070	1990	1910	1840	1770	1710	1650	1590	1540	1490	
			225	0.216	1.163	3940	3680	3450	3240	3060	2900	2750	2610	2490	2380	2270	2180	2090	2000	1930	1860	1790	1730	1670	1610	1560	1510	
	14	6	321	0.252	1.547	4660	4350	4080	3830	3610	3400	3240	3080	2930	2790	2670	2550	2450	2350	2260	2170	2090	2010	1940	1880	1810	1760	
			292	0.252	1.386	4570	4270	4010	3760	3550	3360	3190	3030	2880	2750	2630	2520	2410	2320	2230	2140	2060	1990	1920	1860	1780	1730	
			263	0.252	1.358	4590	4290	4020	3780	3570	3370	3200	3040	2900	2770	2650	2540	2430	2340	2240	2160	2080	2010	1940	1890	1820	1760	
	16	6	367	0.288	1.768	5320	4970	4650	4380	4130	3900	3700	3510	3340	3190	3050	2920	2800	2680	2580	2480	2390	2300	2220	2140	2070	2000	
			333	0.288	1.583	5230	4880	4580	4310	4060	3840	3640	3460	3300	3150	3010	2880	2760	2650	2550	2450	2360	2280	2200	2120	2050	1990	
			300	0.288	1.550	5240	4900	4590	4320	4070	3850	3660	3480	3310	3160	3020	2900	2780	2670	2570	2470	2380	2300	2220	2150	2080	1910	
	18	6	413	0.324	1.990	5990	5590	5240	4920	4640	4390	4160	3950	3760	3590	3430	3280	3140	3010	2900	2790	2680	2590	2500	2410	2330	2250	
			375	0.324	1.780	5890	5500	5160	4850	4580	4330	4100	3900	3710	3540	3390	3240	3110	2980	2870	2760	2660	2560	2480	2390	2310	2240	
			338	0.324	1.745	5900	5510	5160	4860	4580	4340	4110	3910	3730	3560	3400	3260	3120	3000	2890	2780	2680	2580	2500	2410	2330	2260	
30	26.75	10	6	250	0.180	1.070	3620	3380	3160	2970	2810	2650	2510	2390	2270	2170	2070	1980	1900	1820	1750	1680	1620	1560	1510	1460	1410	1360
				229	0.180	0.936	3550	3320	3110	2920	2760	2610	2470	2350	2240	2130	2040	1950	1870	1800	1730	1660	1600	1540	1490	1440	1390	1350
				208	0.180	0.899	3540	3310	3100	2920	2750	2600	2470	2350	2240	2140	2040	1950	1870	1800	1730	1670	1610	1550	1500	1440	1400	1350
	12	6	300	0.180	0.899	3560	3330	3120	2940	2770	2620	2490	2370	2260	2160	2060	1970	1890	1820	1750	1690	1630	1570	1520	1460	1420	1370	
			275	0.216	1.282	4340	4050	3800	3570	3360	3180	3020	2870	2730	2600	2490	2380	2280	2190	2100	2020	1950	1880	1810	1750	1690	1630	
			250	0.216	1.123	4270	3980	3730	3510	3310	3130	2970	2820	2690	2560	2450	2340	2250	2150	2070	1990	1920	1850	1790	1730	1670	1620	
	14	6	350	0.216	1.078	4250	3970	3720	3500	3310	3130	2970	2820	2690	2560	2450	2350	2250	2160	2080	2000	1930	1860	1800	1740	1680	1630	
			225	0.216	1.078	4280	4000	3750	3530	3330	3150	2990	2850	2710	2590	2480	2380	2280	2190	2100	2030	1960	1890	1820	1760	1710	1650	
			300	0.252	1.496	5060	4730	4430	4160	3930	3710	3520	3340	3180	3030	2900	2770	2660	2550	2450	2360	2270	2190	2110	2040	1970	1910	
	16	6	400	0.252	1.310	4970	4640	4350	4090	3850	3650	3460	3290	3130	2990	2850	2730	2620	2510	2420	2320	2240	2160	2080	2010	1950	1880	
			321	0.252	1.257	4960	4630	4340	4080	3850	3640	3460	3290	3130	2990	2860	2740	2620	2520	2420	2330	2250	2170	2090	2020	1960	1890	
			292	0.252	1.257	4990	4660	4320	4110	3880	3670	3490	3320	3160	3020	2890	2770	2650	2550	2450	2360	2280	2200	2120	2050	1990	1920	
	18	6	450	0.288	1.710	5790	5400	5060	4760	4490	4240	4020	3820	3640	3470	3310	3170	3040	2910	2800	2690	2590	2500	2410	2330	2250	2180	
			367	0.288	1.498	5680	5300	4970	4670	4410	4170	3950	3760	3580	3410	3260	3120	2990	2870	2760	2660	2560	2470	2380	2300	2230	2150	
			333	0.288	1.438	5670	5290	4970	4670	4410	4170	3950	3760	3580	3420	3270	3130	3000	2880	2770	2670	2570	2480	2390	2310	2240	2170	
		12	400	0.288	1.438	5700	5320	5000	4700	4440	4200	3990	3790	3610	3450	3300	3160	3030	2910	2800	2700	2600	2500	2430	2350	2270	2200	
			367	0.288	1.438	5700	5320	5000	4700	4440	4200	3990	3790	3610	3450	3300	3160	3030	2910	2800	2700	2600	2500	2430	2350	2270	2200	
			300	0.288	1.438	5700	5320	5000	4700	4440	4200	3990	3790	3610	3450	3300	3160	3030	2910	2800	2700	2600	2500	2430	2350	2270	2200	
		12	450	0.324	1.923	6500	6080	5700	5350	5050	4770	4520	4300	4090	3900	3730	3570	3420	3280	3150	3030	2920	2810	2720	2620	2540	2450	
			413	0.324	1.684	6390	5970	5590	5260	4960	4690	4450	4230	4030	3840	3670	3510	3370	3230	3110	2990	2880	2780	2680	2590	2500	2420	
			375	0.324	1.616	6370	5950	5580	5250	4950	4680	4450	4230	4030	3850	3680	3520	3380	3240	3120	3000	2890	2790	2690	2600	2520	2440	
		12	450	0.324	1.616	6400	5990	5610	5280	4990	4720	4480	4260	4060	3880	3710	3560	3410	3280	3150	3030	2920	2820	2730	2640	2550	2470	



$f_s=20,000$  p.s.i.  
 $f_c=900$  p.s.i.  
 $n=15$

TABLE No. 14 — CONCRETE T-BEAMS

TWO SPANS  
 $M = -\frac{1}{8}wl^2$   
at center support

SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																												
					SPANS IN FEET (l)																							
Total Depth of Beam In.	Effective Depth In.	Width of Web In.	Weight per Foot of Beam Lb.	Required Area of Steel Sq. In.	Reinforcing Steel			6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	
					Straight In.	Bent In.	c <sub>1</sub> (A <sub>s</sub> =c <sub>1</sub> l)																					
12	10	6	75	0.55	1-5/8φ	1-5/8φ	0.108	1670	1210	910	700	550	450	360	300	250	210	170	140	120								
		8	100	0.73	1-5/8φ	1-5/8φ	0.144	2230	1610	1210	940	740	590	480	400	330	270	230	190	160								
		10	125	0.91	1-3/4φ	1-3/4φ	0.180	2790	2020	1510	1170	920	740	600	500	410	340	290	240	200								
14	12	8	117	0.87	1-3/4φ	1-3/4φ	0.144	3230	2340	1770	1370	1090	880	720	600	500	420	350	300	260	220	190	160					
		10	146	1.09	1-3/4φ	1-7/8φ	0.180	4040	2930	2210	1720	1360	1100	900	750	620	530	440	380	320	270	230	200					
		12	175	1.31	1-7/8φ	1-1φ	0.216	4850	3520	2650	2060	1640	1320	1080	900	750	630	530	450	380	330	280	240					
16	14	8	133	1.02	1-3/4φ	1-7/8φ	0.144	3750	3210	2430	1890	1510	1220	1010	840	700	600	510	430	370	320	280	240					
		10	167	1.27	1-7/8φ	1-1φ	0.180	4680	4010	3030	2360	1890	1530	1260	1050	880	750	640	540	470	400	350	300					
		12	200	1.53	1-1φ	1-1φ	0.216	5620	4820	3640	2830	2260	1830	1510	1250	1060	890	760	650	560	480	420	360					
18	16	8	150	1.16	1-7/8φ	1-7/8φ	0.144	4280	3650	3200	2500	2000	1620	1340	1120	940	810	690	590	510	450	390	340					
		10	188	1.46	1-1φ	1-1φ	0.180	5360	4560	4000	3120	2490	2020	1670	1400	1180	1000	860	740	640	560	480	420					
		12	225	1.75	2-3/4φ	2-3/4φ	0.216	6430	5480	4810	3750	3000	2440	2010	1680	1420	1210	1030	890	770	670	580	510					
		14	262	2.04	2-3/4φ	2-7/8φ	0.252	7510	6390	5590	4360	3490	2840	2340	1960	1650	1400	1200	1040	900	780	680	590					
20	18	8	167	1.31	1-7/8φ	1-1φ	0.144	4820	4100	3570	3180	2550	2070	1720	1440	1220	1040	890	770	670	590	510	450					
		10	208	1.64	2-3/4φ	2-3/4φ	0.180	6020	5130	4460	3980	3190	2600	2150	1800	1530	1300	1120	970	840	740	640	560					
		12	250	1.97	2-3/4φ	2-7/8φ	0.216	7230	6150	5350	4780	3830	3120	2580	2160	1830	1560	1340	1160	1010	880	770	670					
		14	292	2.29	2-7/8φ	2-7/8φ	0.252	8440	7180	6260	5560	4460	3630	3010	2520	2130	1820	1560	1350	1170	1020	900	790					
22	20	10	223	1.82	1-1φ	1-1φ	0.180					3930	3230	2680	2250	1910	1630	1410	1220	1060	930	820	720					
		12	275	2.18	2-7/8φ	2-7/8φ	0.216					4720	3880	3220	2700	2290	1960	1690	1470	1280	1120	980	870					
		14	321	2.55	2-7/8φ	2-1φ	0.252					5500	4530	3760	3150	2670	2290	1970	1710	1490	1310	1150	1010					
		16	367	2.91	2-1φ	2-1φ	0.288					6290	5170	4280	3590	3050	2610	2250	1950	1700	1490	1310	1150					
24	22	10	250	2.00	1-1φ	1-1φ	0.180					4330	3910	3270	2750	2340	2010	1730	1510	1320	1160	1020	900					
		12	300	2.40	2-7/8φ	2-7/8φ	0.216					5190	4690	3920	3290	2800	2400	2080	1800	1580	1380	1220	1080					
		14	350	2.80	2-7/8φ	2-1φ	0.252					6050	5480	4580	3850	3270	2810	2420	2110	1840	1620	1430	1260					
		16	400	3.20	2-1φ	2-1φ	0.288					6920	6260	5250	4400	3750	3210	2780	2410	2110	1850	1630	1440					
26	24	10	271	2.18	1-1φ	2-1φ	0.180					4720	4260	3880	3300	2810	2410	2090	1820	1590	1400	1240	1100					
		12	325	2.62	2-7/8φ	2-1φ	0.216					5670	5130	4670	3970	3380	2900	2510	2190	1920	1690	1490	1320					
		14	379	3.06	2-1φ	2-1φ	0.252					6610	5970	5440	4620	3930	3380	2920	2540	2230	1960	1730	1530					
		16	433	3.49	2-1φ	2-1φ	0.288					7560	6830	6220	5280	4500	3860	3340	2910	2550	2250	1980	1760					
28	26	12	350	2.84	2-1φ	2-1φ	0.216					6140	5550	5050	4640	4000	3440	2980	2600	2280	2010	1780	1580					
		14	408	3.31	2-1φ	2-1φ	0.252					7160	6480	5900	5410	4650	4000	3460	3020	2650	2340	2070	1840					
		16	467	3.78	2-1φ	2-1φ	0.288					8180	7390	6730	6180	5320	4570	3960	3450	3030	2670	2360	2100					
		18	525	4.26	3-7/8φ	4-7/8φ	0.324					9210	8330	7590	6960	5980	5140	4450	3890	3410	3010	2660	2370					
30	28	12	375	3.06	2-1φ	2-1φ	0.216							5450	5000	4630	4010	3480	3040	2670	2360	2090	1860					
		14	437	3.57	2-1φ	2-1φ	0.252							6350	5820	5380	4670	4050	3540	3110	2750	2440	2170					
		16	500	4.08	3-7/8φ	3-1φ	0.288							7270	6670	6150	5340	4630	4050	3560	3140	2790	2480					
		18	563	4.59	3-1φ	3-1φ	0.324							8170	7500	6930	6010	5210	4550	4000	3530	3130	2790					
		20	625	5.10	3-1φ	4-1φ	0.360							9080	8330	7680	6680	5790	5060	4450	3930	3480	3100					

\*For all loadings to the right of the heavy zig-zag line, use the area of steel shown in the table. Loads to the left of the heavy line are governed by shear ( $v=120$ ) and the areas of steel must be computed by means of the coefficient  $c_1$  as indicated.



$f_s = 20,000$  p.s.i.  
 $f_c = 900$  p.s.i.  
 $n = 15$

TABLE No. 15 — CONCRETE T-BEAMS

END SPANS (More than 2 spans)

$M = -\frac{1}{8} w l^2$

SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																											
SPANS IN FEET (l)																											
Total Depth of Beam In.	Effective Depth In.	Width of Web In.	Weight per Foot of Beam Lb.	Re-quired Area of Steel Sq. In.	Reinforcing Steel			6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
					Straight In.	Bent In.	$c_1$ ( $A_s = c_1 I$ )																				
12	10	6	75	0.55	1- $\frac{5}{8}\phi$	1- $\frac{5}{8}\phi$	0.086	2010	1530	1150	900	710	580	470	390	330	270	230	200	170							
		8	100	0.73	1- $\frac{5}{8}\phi$	1- $\frac{3}{4}\phi$	0.115	2670	2040	1540	1190	950	770	630	520	440	370	310	260	220							
		10	125	0.91	1- $\frac{3}{4}\phi$	1- $\frac{3}{4}\phi$	0.144	3340	2570	1920	1490	1190	960	790	650	540	460	390	330	280							
14	12	8	117	0.87	1- $\frac{3}{4}\phi$	1- $\frac{3}{4}\phi$	0.115	3210	2740	2240	1750	1390	1130	930	780	650	550	470	410	350	300	260	230	280	340		
		10	146	1.09	1- $\frac{3}{4}\phi$	1- $\frac{7}{8}\phi$	0.144	4010	3410	2800	2180	1740	1410	1160	970	820	690	590	510	440	380	330	280	340			
		12	175	1.31	1- $\frac{7}{8}\phi$	1-1 $\phi$	0.173	4820	4100	3360	2620	2090	1690	1400	1160	980	830	710	610	520	450	390	340				
16	14	8	133	1.02	1- $\frac{3}{4}\phi$	1- $\frac{7}{8}\phi$	0.115	3750	3200	2780	2400	1920	1570	1290	1080	920	780	670	580	500	440	380	330	330	420		
		10	167	1.27	1- $\frac{7}{8}\phi$	1-1 $\phi$	0.144	4680	3990	3470	3000	2400	1950	1620	1350	1140	970	840	720	630	550	480	420	500			
		12	200	1.53	1-1 $\phi$	1-1 $\phi$	0.173	5620	4790	4170	3600	2880	2350	1940	1620	1370	1170	1000	870	750	650	570	500				
18	16	8	150	1.16	1- $\frac{7}{8}\phi$	1- $\frac{7}{8}\phi$	0.115	4280	3650	3170	2810	2540	2070	1720	1440	1220	1040	900	780	680	590	520	460	570			
		10	188	1.46	1-1 $\phi$	1-1 $\phi$	0.144	5360	4560	3970	3510	3160	2580	2140	1790	1520	1300	1120	970	850	740	650	570				
		12	225	1.75	2- $\frac{3}{4}\phi$	2- $\frac{3}{4}\phi$	0.173	6430	5480	4770	4220	3800	3100	2570	2160	1830	1560	1350	1170	1020	890	780	690	800			
		14	262	2.04	2- $\frac{3}{4}\phi$	2- $\frac{7}{8}\phi$	0.202	7510	6390	5560	4920	4430	3610	2990	2510	2130	1820	1570	1360	1190	1040	910	800				
20	18	8	167	1.31	1- $\frac{7}{8}\phi$	1-1 $\phi$	0.115	4820	4100	3570	3160	2830	2550	2190	1840	1560	1340	1160	1010	880	770	680	600	750			
		10	208	1.64	2- $\frac{3}{4}\phi$	2- $\frac{3}{4}\phi$	0.144	6020	5130	4460	3950	3530	3190	2730	2300	1950	1680	1450	1260	1100	970	850	750				
		12	250	1.97	2- $\frac{3}{4}\phi$	2- $\frac{7}{8}\phi$	0.173	7230	6150	5350	4740	4240	3840	3290	2770	2350	2020	1740	1510	1320	1160	1030	910	1060			
		14	292	2.29	2- $\frac{7}{8}\phi$	2- $\frac{7}{8}\phi$	0.202	8440	7180	6260	5530	4950	4470	3840	3230	2750	2350	2030	1770	1540	1360	1200	1060				
22	20	10	229	1.82	1-1 $\square$	1-1 $\square$	0.144					3930	3560	3240	2870	2440	2100	1820	1580	1390	1220	1080	960	850	760	680	610
		12	275	2.18	2- $\frac{7}{8}\phi$	2- $\frac{7}{8}\phi$	0.173					4720	4270	3890	3450	2940	2520	2180	1900	1670	1470	1300	1150	1030	910	820	730
		14	321	2.55	2- $\frac{7}{8}\phi$	2-1 $\phi$	0.202					5500	4980	4540	4020	3420	2940	2540	2220	1940	1710	1510	1340	1200	1070	950	850
		16	367	2.91	2-1 $\phi$	2-1 $\phi$	0.230					6290	5680	5180	4590	3910	3360	2910	2530	2220	1950	1730	1530	1370	1220	1090	970
24	22	10	250	2.00	1-1 $\square$	1-1 $\square$	0.144					4330	3910	3570	3270	2990	2570	2230	1940	1710	1510	1330	1190	1060	950	850	760
		12	300	2.40	2- $\frac{7}{8}\phi$	2- $\frac{7}{8}\phi$	0.173					5190	4690	4280	3920	3580	3080	2670	2330	2050	1810	1600	1420	1270	1140	1020	920
		14	350	2.80	2- $\frac{7}{8}\phi$	2-1 $\phi$	0.202					6050	5480	4990	4570	4180	3590	3120	2720	2390	2110	1870	1660	1480	1330	1190	1070
		16	400	3.20	2-1 $\phi$	2-1 $\phi$	0.230					6920	6260	5700	5240	4780	4110	3570	3120	2740	2410	2140	1900	1700	1520	1360	1220
26	24	10	271	2.18	1-1 $\phi$	2-1 $\phi$	0.144					4720	4270	3880	3570	3290	3050	2680	2340	2060	1820	1610	1440	1290	1160	1040	940
		12	325	2.62	2- $\frac{7}{8}\phi$	2-1 $\phi$	0.173					5670	5130	4670	4290	3960	3670	3210	2810	2470	2180	1940	1730	1550	1390	1250	1120
		14	379	3.06	2-1 $\phi$	2-1 $\phi$	0.202					6610	5970	5440	5000	4610	4280	3740	3270	2880	2540	2260	2010	1800	1620	1450	1310
		16	433	3.49	2-1 $\phi$	2-1 $\phi$	0.230					7560	6830	6220	5720	5270	4890	4290	3750	3300	2920	2590	2310	2070	1850	1670	1500
28	26	12	350	2.84	2-1 $\phi$	2-1 $\phi$	0.173					6140	5550	5050	4640	4290	3980	3710	3330	2940	2600	2310	2060	1850	1660	1500	1350
		14	408	3.31	2-1 $\phi$	2-1 $\phi$	0.202					7160	6480	5900	5410	5000	4640	4320	3880	3420	3030	2690	2400	2160	1940	1750	1580
		16	467	3.78	2-1 $\phi$	2-1 $\phi$	0.230					8180	7390	6730	6180	5710	5290	4940	4430	3900	3450	3070	2740	2460	2210	1960	1800
		18	525	4.26	3- $\frac{7}{8}\phi$	4- $\frac{7}{8}\phi$	0.259					9210	8330	7590	6960	6430	5960	5550	4990	4390	3890	3500	3090	2770	2490	2240	2030
30	28	12	375	3.06	2-1 $\phi$	2-1 $\phi$	0.173																				
		14	437	3.57	2-1 $\phi$	2-1 $\phi$	0.202																				
		16	500	4.08	3- $\frac{7}{8}\phi$	3-1 $\phi$	0.230																				
		18	563	4.59	3-1 $\phi$	3-1 $\phi$	0.259																				
		20	625	5.10	3-1 $\phi$	4-1 $\phi$	0.288																				



$f_s=20,000$  p.s.i.  
 $f_c=900$  p.s.i.  
 $n=15$

TABLE No. 16—CONCRETE T-BEAMS

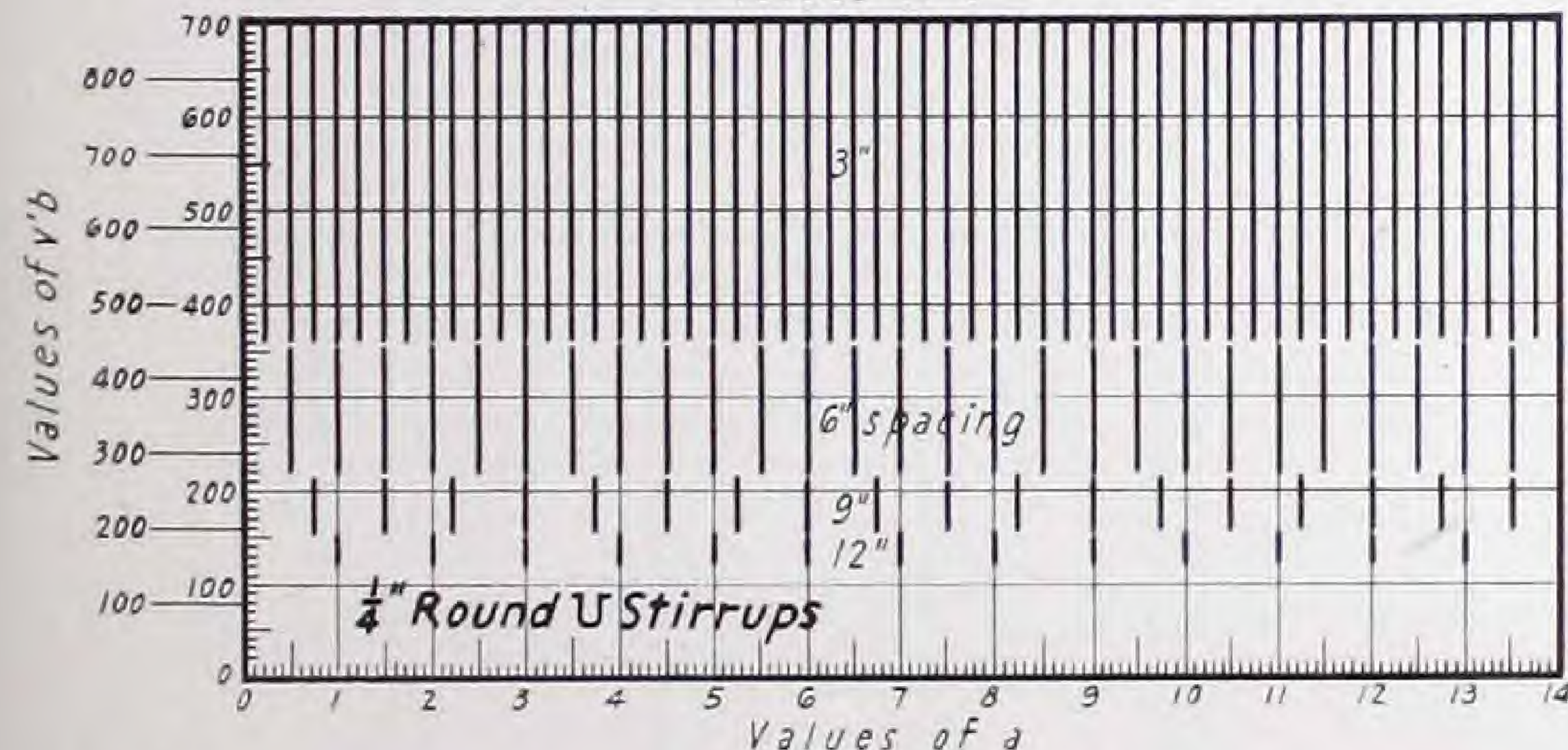
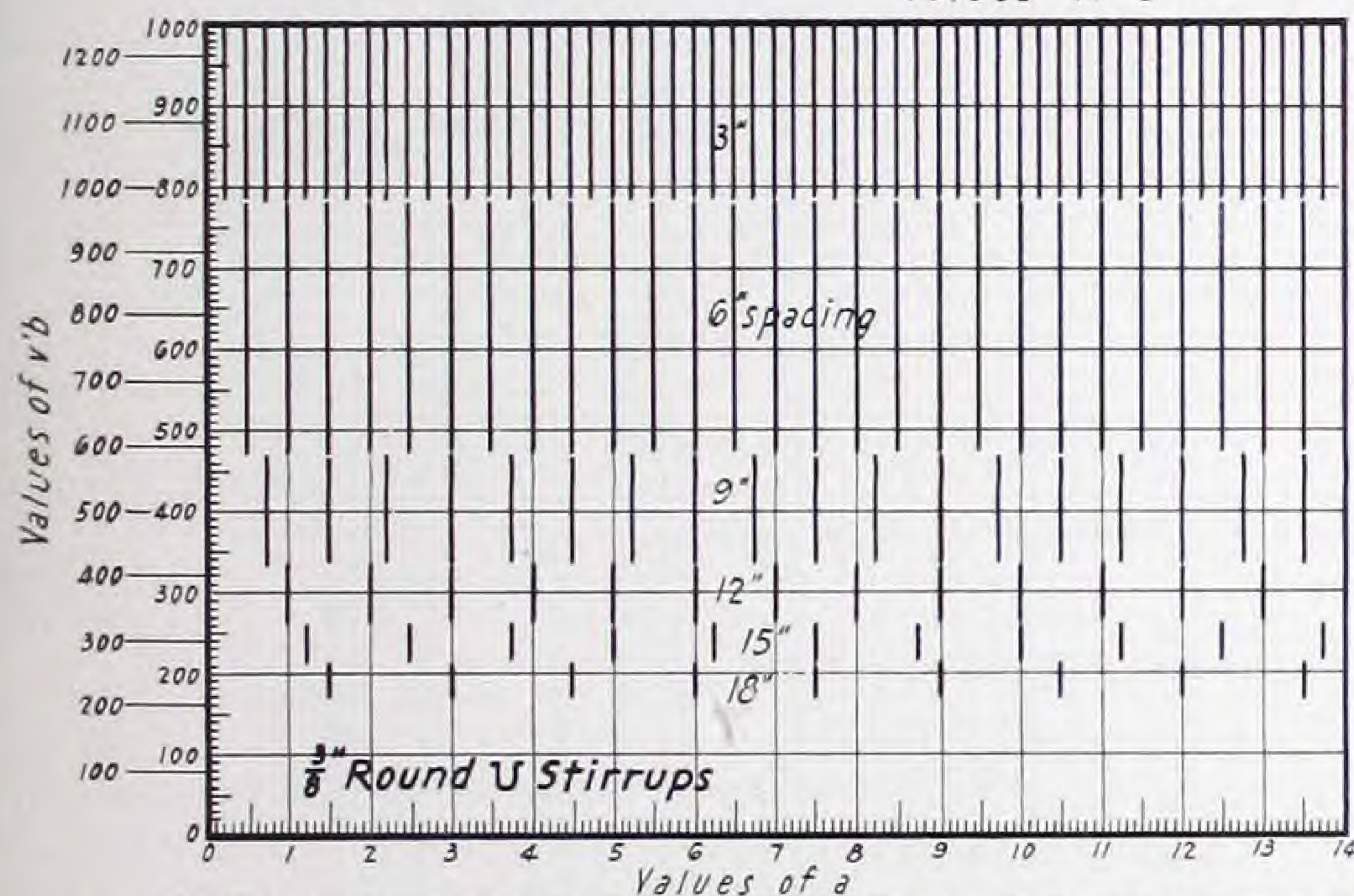
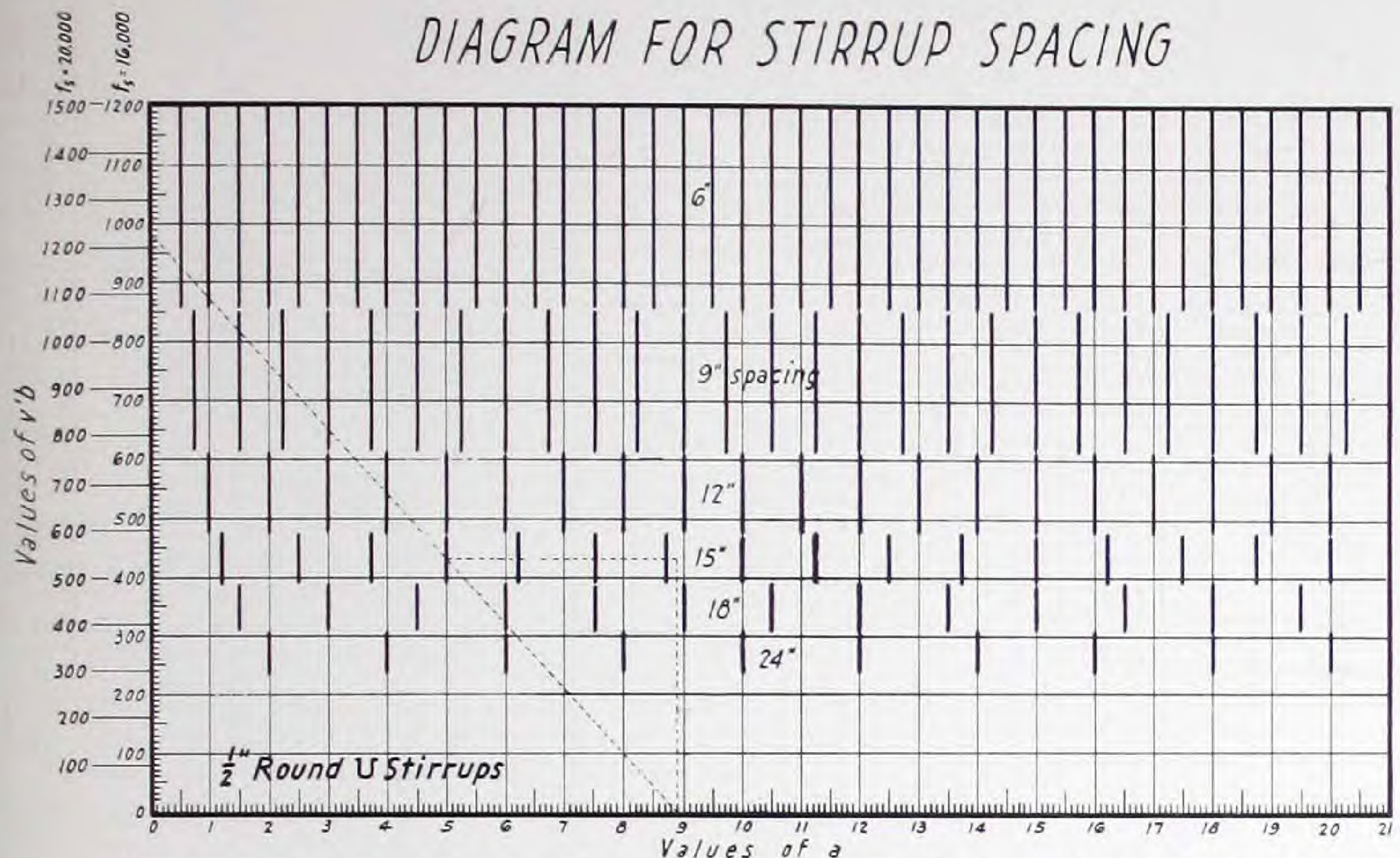
INTERIOR SPANS  
 $M=-\frac{1}{2}wl^2$

SAFE SUPERIMPOSED LOADS IN LB. PER FT. OF BEAM																											
					SPANS IN FEET ( <i>l</i> )																						
					6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25			
Total Depth of Beam In.	Effective Depth In.	Width of Web In.	Weight per Foot of Beam Lb.	Re-quired Area of Steel Sq. In.	Reinforcing Steel		$c_1$ ( $A_s=c_1l$ )																				
					Straight In.	Bent In.																					
12	10	6 8 10	75 100 125	0.55 0.73 0.91	1- $\frac{5}{8}\phi$ 1- $\frac{5}{8}\phi$ 1- $\frac{3}{4}\phi$	1- $\frac{5}{8}\phi$ 1- $\frac{3}{4}\phi$ 1- $\frac{3}{4}\phi$	0.072 0.096 0.120	2010 2670 3340	1710 2280 2850	1400 1860 2330	1090 1450 1820	870 1160 1450	700 940 1170	580 770 970	480 640 810	410 540 680	340 460 570	290 390 490	250 340 420	220 290 360							
14	12	8 10 12	117 146 175	0.87 1.09 1.31	1- $\frac{3}{4}\phi$ 1- $\frac{3}{4}\phi$ 1- $\frac{7}{8}\phi$	1- $\frac{3}{4}\phi$ 1- $\frac{7}{8}\phi$ 1-1 $\phi$	0.096 0.120 0.144	3210 4010 4820	2740 3410 4100	2380 2970 3570	2120 2640 3170	1690 2110 2540	1380 1720 2070	1140 1420 1710	950 1190 1430	810 1010 1210	690 860 1030	590 740 880	510 640 760	440 550 660	380 480 580	340 420 500	290 370 440				
16	14	8 10 12	133 167 200	1.02 1.27 1.53	1- $\frac{3}{4}\phi$ 1- $\frac{7}{8}\phi$ 1-1 $\phi$	1- $\frac{7}{8}\phi$ 1-1 $\phi$ 1-1 $\phi$	0.096 0.120 0.144	3750 4680 5620	3200 3990 4790	2780 3470 4170	2460 3070 3680	2200 2740 3300	1900 2380 2850	1580 1970 2360	1320 1660 1980	1120 1400 1680	960 1200 1440	830 1040 1240	720 900 1080	630 780 940	550 690 820	480 600 720	420 530 640				
18	16	8 10 12 14	150 188 225 262	1.16 1.46 1.75 2.04	1- $\frac{7}{8}\phi$ 1-1 $\phi$ 2- $\frac{3}{4}\phi$ 2- $\frac{3}{4}\phi$	1- $\frac{7}{8}\phi$ 1-1 $\phi$ 2- $\frac{3}{4}\phi$ 2- $\frac{7}{8}\phi$	0.096 0.120 0.144 0.168	4280 5360 6430 7510	3650 4560 5480 6390	3170 3970 4770 5560	2810 3510 4210 4920	2510 3140 3770 4400	2270 2840 3410 3980	2070 2590 3110 3620	1760 2190 2630 3060	1490 1860 2240 2610	1280 1600 1920 2240	1110 1380 1660 1940	960 1200 1440 1680	850 1050 1260 1480	740 930 1110 1300	660 820 980 1140	580 720 870 1010				
20	18	8 10 12 14	167 208 250 292	1.31 1.64 1.97 2.29	1- $\frac{7}{8}\phi$ 2- $\frac{3}{4}\phi$ 2- $\frac{3}{4}\phi$ 2- $\frac{7}{8}\phi$	1-1 $\phi$ 2- $\frac{3}{4}\phi$ 2- $\frac{7}{8}\phi$ 2- $\frac{7}{8}\phi$	0.096 0.120 0.144 0.168	4820 6020 7230 8440	4100 5130 6150 7180	3570 4460 5350 6260	3160 3950 4740 5530	2830 3530 4240 4950	2550 3190 3840 4470	2330 2910 3490 4070	2140 2670 3200 3740	1910 2380 2860 3340	1640 2050 2460 2870	1420 1770 2130 2490	1240 1550 1860 2170	1090 1360 1630 1910	960 1200 1440 1680	850 1060 1270 1490	760 940 1130 1320				
22	20	10 12 14 16	229 275 321 367	1.82 2.18 2.55 2.91	1-1 $\square$ 2- $\frac{7}{8}\phi$ 2- $\frac{7}{8}\phi$ 2-1 $\phi$	1-1 $\square$ 2- $\frac{7}{8}\phi$ 2-1 $\phi$ 2-1 $\phi$	0.120 0.144 0.168 0.192					3930 4720 5500 6290	3560 4270 4980 5680	3240 3890 4540 5180	2970 3570 4160 4750	2740 3290 3840 4390	2550 3050 3560 4070	2220 2670 3110 3560	1940 2340 2720 3110	1710 2050 2390 2730	1510 1810 2120 2410	1340 1610 1880 2140	1190 1430 1670 1910	1070 1280 1500 1720	960 1150 1340 1530	860 1030 1210 1380	780 930 1090 1240
24	22	10 12 14 16	250 300 350 400	2.00 2.40 2.80 3.20	1-1 $\square$ 2- $\frac{7}{8}\phi$ 2- $\frac{7}{8}\phi$ 2-1 $\phi$	1-1 $\square$ 2- $\frac{7}{8}\phi$ 2-1 $\phi$ 2-1 $\phi$	0.120 0.144 0.168 0.192					4330 5190 6050 6920	3910 4690 5480 6260	3570 4280 4990 5700	3270 3920 4570 5240	3020 3620 4220 4830	2800 3360 3920 4480	2610 3130 3650 4180	2380 2850 3330 3810	2090 2510 2930 3360	1850 2220 2590 2970	1650 1980 2310 2640	1470 1760 2060 2360	1320 1580 1850 2120	1190 1420 1660 1900	1070 1280 1490 1710	970 1160 1350 1550
26	24	10 12 14 16	271 325 379 433	2.18 2.62 3.06 3.49	1-1 $\phi$ 2- $\frac{7}{8}\phi$ 2-1 $\phi$ 2-1 $\phi$	2-1 $\phi$ 2-1 $\phi$ 2-1 $\phi$ 2-1 $\phi$	0.120 0.144 0.168 0.192					4720 5670 6610 7560	4270 5130 5970 6830	3880 4670 5440 6220	3570 4290 5000 5720	3290 3960 4610 5270	3050 3670 4280 4890	2850 3420 3990 4560	2660 3200 3730 4270	2500 3010 3500 4010	2230 2680 3130 3580	1990 2390 2780 3190	1780 2140 2490 2850	1600 1920 2240 2560	1440 1730 2010 2310	1300 1560 1820 2080	1180 1410 1650 1890
28	26	12 14 16 18	350 408 467 525	2.84 3.31 3.78 4.26	2-1 $\phi$ 2-1 $\phi$ 2-1 $\phi$ 3- $\frac{7}{8}\phi$	2-1 $\phi$ 2-1 $\phi$ 2-1 $\phi$ 4- $\frac{7}{8}\phi$	0.144 0.168 0.192 0.216					6140 7160 8180 9210	5550 6480 7390 8330	5050 5900 6730 7590	4640 5410 6180 6960	4290 5000 5710 6430	3980 4640 5290 5960	3710 4320 4940 5550	3470 4040 4620 5200	3260 3800 4340 4880	3070 3580 4080 4600	2840 3300 3780 4250	2540 2960 3380 3800	2290 2660 3050 3420	2060 2400 2750 3090	1870 2170 2480 2790	1690 1970 2250 2530
30	28	12 14 16 18 20	375 437 500 563 625	3.06 3.57 4.08 4.59 5.10	2-1 $\phi$ 2-1 $\phi$ 3- $\frac{7}{8}\phi$ 3-1 $\phi$ 3-1 $\phi$	2-1 $\phi$ 2-1 $\phi$ 3- $\frac{7}{8}\phi$ 3-1 $\phi$ 4-1 $\phi$	0.144 0.168 0.192 0.216 0.240							5450 6350 7270 8170 9080	5000 5820 6670 7500 8330	4630 5380 6150 6930 7700	4290 5000 5710 6420 7130	4000 4650 5330 5990 6650	3740 4360 4990 5600 6220	3510 4090 4680 5260 5840	3310 3850 4410 4950 5500	3120 3640 4160 4680 5190	2960 3440 3940 4430 4920	2680 3120 3570 4010 4470	2420 2820 3230 3620 4030	2190 2550 2920 3280 3650	1990 2320 2650 2980 3320

\*For all loadings to the right of the heavy zig-zag line, use the area of steel shown in the table. Loads to the left of the heavy line are governed by shear ( $v=120$ ) and the areas of steel must be computed by means of the coefficient  $c_1$  as indicated.



# DIAGRAM FOR STIRRUP SPACING



## TO USE THIS CHART

Find the unit shear,  $v = \frac{V}{b \cdot d}$   
 Then  $v' = v - v_c$  and distance  $a$ , over which stirrups are required =  $\frac{1}{2}(\frac{v'}{v_c})$  Compute  $v'b$  Place a straight-edge on the diagram connecting the calculated values of  $a$  and  $v'b$ , and read off the spacing of stirrups where their respective heavy lines cross the straight-edge. In case the spacing is greater than the maximum allowable spacing of  $\frac{1}{2}d$ , use the last spacing permitted, until distance  $a$  is reached. For example:  $l = 28'$ ,  $b = 14"$ ,  $d = 30"$ ,  $V = 40,500\#$ , the beam having a uniformly distributed load. Required the number and spacing of  $\frac{1}{2}" \phi$  U stirrups in each end of the beam,  $f_s = 16,000$ .

$$v = \frac{40,500}{14 \times 0.875 \times 30} = 110 \text{ p.s.i.}$$

$$v' = 110 - 40 = 70 \text{ p.s.i.}$$

$$v'b = 14 \times 70 = 980$$

$$a = \frac{28}{2} \left( \frac{70}{110} \right) = 8.9'$$

$$\text{Max. allowable spacing} = .50 \times 30 = 15"$$

From the chart  
 $s = 6", 6", 3 @ 9", 12", 4 @ 15"$   
 No. of stirrups in each end of beam = 10

Fig. 17



$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
 $\frac{3}{4}$ " clear fireproofing

TABLE No. 17 — TWO-WAY SOLID CONCRETE SLABS

TABLE No. 17 — TWO-WAY SOLID CONCRETE SLABS										SQUARE PANELS																			
$f_s=20,000$ p.s.i. $f_c=800$ p.s.i. $n=15$ $\frac{3}{4}"$ clear fireproofing										SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																			
Total Thickness of Slab		Weight per Sq. Ft. of Slab	Volume of Concrete per Sq. Ft.	Condition of Continuity		Reinforcing Steel Size and Spacing in Inches		SPANS IN FEET																					
				Either Direction	Other Direction	Bottom Layer	Top Layer	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	
4	50	0.333	{ Simple End Interior	{ Simple End Interior	$\frac{3}{8}\phi-6$	$\frac{3}{8}\phi-5\frac{1}{2}$	178 163 222 171 221 291	138 126 175 133 174 232	108 98 139 103 138 187	111 81 110 152	88 124	102																	
4½	56	0.375			$\frac{1}{2}\phi-10$	$\frac{1}{2}\phi-8\frac{1}{2}$	227 210 283 219 281	178 164 224 172 223 295	141 129 180 135 178 239	112 101 144 107 143 196	80 117 84 116 161	95 66 94 133	110																
5	63	0.417			$\frac{1}{2}\phi-8\frac{1}{2}$	$\frac{1}{2}\phi-7\frac{1}{2}$		256 236 247	205 188 257 197 257	165 151 210 159 209 279	134 122 172 128 172 232	108 98 142 103 141 194	117 83 117 163																
5½	69	0.458			$\frac{1}{2}\phi-7\frac{1}{2}$	$\frac{1}{2}\phi-6\frac{1}{2}$			281 259 271	229 211 287 221 286	188 172 238 181 237	155 141 199 149 198 267	128 115 166 122 165 226	96 137	115														
6	75	0.500			$\frac{1}{2}\phi-6\frac{1}{2}$	$\frac{1}{2}\phi-5\frac{1}{2}$				289 292	251 230 241	208 191 264 201 263	174 159 223 167 220	146 132 189 139 256	122 109 160 116 221	97 140	120												
6½	81	0.542			$\frac{1}{2}\phi-5\frac{1}{2}$	$\frac{1}{2}\phi-5$						269 247 259	227 208 287 218	191 175 245 184	162 147 210 155	137 123 180 131	104 154 110	122 92	123	97 142	115 164	135 190	115 164						







$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
 $\frac{3}{4}$ " clear fireproofing

TABLE No. 18 — TWO-WAY SOLID CONCRETE SLABS

RATIO OF SPANS  
1.2 to 1.0

Total Thickness of Slab In.	Weight per Sq. Ft. of Slab Lb.	Volume of Concrete per Sq. Ft. Cu. Ft.	Condition of Continuity		Reinforcing Steel Size and Spacing in Inches		SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
			Short Way of Slab	Long Way of Slab	ShortWay (Bottom Layer)	Long Way (Top Layer)	SPANS IN FEET																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
							12	13.2	14.4	15.6	16.8	18	19.2	20.4	21.6	22.8	24	25.2	26.4	27.6	28.8	30																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
4	50	0.333	Simple End Interior	Simple End Interior	$\frac{3}{8}\phi-4\frac{1}{2}$	$\frac{3}{8}\phi-5\frac{1}{2}$	169																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										



[illegible]











$f_s=20,000$  p.s.i.  
 $f_c=800$  p.s.i.  
 $n=15$   
 $\frac{3}{4}$ " clear fireproofing

TABLE No. 20  
TWO-WAY SOLID CONCRETE SLABS

RATIO OF  
SPANS  
1.6 to 1.0

Total Thickness of Slab In.	Weight per Sq. Ft. Slab Lb.	Volume of Concrete per Sq. Ft. Cu. Ft.	Condition of Continuity		Reinforcing Steel Size and Spacing in Inches	SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.											
						SPANS IN FEET											
			Short Way of Slab	Long Way of Slab		Short Way (Bottom Layer)	Long Way (Top Layer)	16 x 10	17.6 x 11	19.2 x 12	20.8 x 13	22.4 x 14	24 x 15	25.6 x 15	27.2 x 17	28.8 x 18	30 x 18.75
4	50	0.333	Simple End Interior	Interior	3/8φ-5 1/2	146											
					3/8φ-8 1/2	126											
					3/8φ-6 1/2	160	124										
					3/8φ-10	114											
					3/8φ-10	137											
					3/8φ-9	170	132										
4 1/2	56	0.375	Simple End Interior	Simple End Interior	3/8φ-8	136											
					1 1/2φ-10	175	135										
					1 1/2φ-8 1/2	190	147										
					3/8φ-10	137											
					3/8φ-7 1/2	173	133										
					3/8φ-6	218	170	134									
5	63	0.417	Simple End Interior	Simple End Interior	3/8φ-10	158	121										
					3/8φ-10	188	146	113									
					3/8φ-8	232	182	144	114								
					1 1/2φ-7 1/2												
					1 1/2φ-6 1/2	188	144										
					1 1/2φ-6 1/2	239	186	147	135								
5 1/2	69	0.458	Simple End Interior	Simple End Interior	1 1/2φ-7 1/2	272	234	170									
					1 1/2φ-6 1/2	190	146	102									
					3/8φ-9 1/2	237	185	145									
					3/8φ-6 1/2	295	233	185	149								
					3/8φ-5	217	168	131									
					3/8φ-9 1/2	256	200	158	126	129							
6	75	0.500	Simple End Interior	Simple End Interior	3/8φ-7	313	247	198	159								
					3/8φ-6	248	193	151									
					1 1/2φ-7 1/2	313	247	197	157								
					1 1/2φ-6 1/2	251	195	153	190	154							
					3/8φ-8 1/2	310	244	194	155	124	133						
					3/8φ-6	285	306	246	199	162							
6	75	0.500	Simple End Interior	Simple End Interior	3/8φ-4 1/2		223	177	140	137	142						
					3/8φ-5 1/2		265	211	170	173							
					1 1/2φ-5 1/2			261	212								
					1 1/2φ-5	241	190	156									
					1 1/2φ-5		253	204	166								
					1 1/2φ-5		253	204	207	171							
6	75	0.500	Simple End Interior	Simple End Interior	3/8φ-7 1/2	199	199	159	126								
					3/8φ-5	251	251	202	154	123							
					1 1/2φ-4			256	211	164							
					1 1/2φ-4			256	211	164							
					1 1/2φ-4			256	211	164							
					1 1/2φ-4			256	211	164							



61 81 0.542 7 88 0.583 7 1/2 94 0.625 8 100 0.667 8 1/2 106 0.708 9 113 0.750

6 1/2	81	0.542	Simple End Interior	5/8 φ-7 1/2 5/8 φ-7 1/2 5/8 φ-7 1/2 5/8 φ-6 5/8 φ-6 5/8 φ-7 1/2 5/8 φ-7 1/2 5/8 φ-7 1/2	1 1/2 φ-9 5/8 φ-9 1/2 5/8 φ-8 5/8 φ-7 1 1/2 φ-8 1/2 1 1/2 φ-7 3/8 φ-7 3/8 φ-7 1 1/2 φ-9	302	241	193	155	125	137	138	111
7	88	0.583	Simple End Interior	5/8 φ-6 1/2 5/8 φ-6 1/2 5/8 φ-6 1/2 5/8 φ-5 1/2 5/8 φ-5 1/2 5/8 φ-6 1/2 5/8 φ-6 1/2 5/8 φ-6 1/2	1 1/2 φ-8 1/2 5/8 φ-9 5/8 φ-7 1/2 3/8 φ-6 1/2 1 1/2 φ-8 1 1/2 φ-6 1/2 3/8 φ-6 1/2 3/8 φ-6 1/2 1 1/2 φ-8 1/2	296	289	239	194	158	128	142	150
7 1/2	94	0.625	Simple End Interior	5/8 φ-6 5/8 φ-6 5/8 φ-6 5/8 φ-5 5/8 φ-5 5/8 φ-6 5/8 φ-6 5/8 φ-6	1 1/2 φ-8 5/8 φ-8 5/8 φ-7 3/8 φ-6 1 1/2 φ-7 1 1/2 φ-6 3/8 φ-6 3/8 φ-6 1 1/2 φ-8			290	237	195	159	130	168
8	100	0.667	Simple End Interior	5/8 φ-5 1/2 5/8 φ-5 1/2 5/8 φ-5 1/2 5/8 φ-4 1/2 5/8 φ-4 1/2 5/8 φ-5 1/2 5/8 φ-5 1/2 5/8 φ-5 1/2	1 1/2 φ-7 1/2 5/8 φ-7 1/2 5/8 φ-6 1 1/2 φ-10 1 1/2 φ-6 1/2 1 1/2 φ-5 1/2 1 1/2 φ-10 1 1/2 φ-10 1 1/2 φ-7 1/2				285	235	161	133	159
8 1/2	106	0.708	Simple End Interior	3/4 φ-8 3/4 φ-8 3/4 φ-8 3/4 φ-6 1/2 3/4 φ-6 1/2 3/4 φ-6 1/2 3/4 φ-8 3/4 φ-8	1 1/2 φ-7 3/4 φ-10 3/4 φ-8 1/2 1 1/2 φ-9 1 1/2 φ-6 1 1/2 φ-5 1 1/2 φ-9 1 1/2 φ-9					273	189	157	136
9	113	0.750	Simple End Interior	3/4 φ-7 1/2 3/4 φ-7 1/2 3/4 φ-7 1/2 3/4 φ-6 3/4 φ-6 3/4 φ-6 3/4 φ-7 1/2 3/4 φ-7 1/2	1 1/2 φ-6 1/2 3/4 φ-9 1/2 3/4 φ-8 1 1/2 φ-8 1/2 1 1/2 φ-6 1 1/2 φ-5 1 1/2 φ-8 1/2 1 1/2 φ-6 1/2					267	223	187	164

1-r For Shear	
For Long Beam	For Short Beam
0.804	0.196
0.730	0.270
0.643	0.357
0.862	0.138
0.804	0.196
0.732	0.268
0.904	0.096
0.860	0.140
0.804	0.196

1-r For Bending	
For Long Beam	For Short Beam
0.880	0.330
0.833	0.445
0.775	0.540
0.920	0.205
0.880	0.330
0.835	0.440
0.944	0.098
0.918	0.210
0.880	0.330



TABLE No. 21—TWO-WAY SOLID CONCRETE SLABS										RATIO OF SPANS 1.8 to 1.0						
SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.			SPANS IN FEET													
Total Thick- ness of Slab In.	Weight per Sq. Ft. Slab Lb.	Volume of Concrete per Sq. Ft. Cu. Ft.	Condition of Continuity		Reinforcing Steel Size and Spacing in Inches		18 x 10	19.8 x 11	21.6 x 12	23.4 x 13	25.2 x 14	27 x 15	28.8 x 16	30 x 16.7		
			Short Way of Slab	Long Way of Slab	Short Way (Bottom Layer)	Long Way (Top Layer)										
4½	56	0.375	Simple	End	1½φ-7½	¾φ-6½	140	130								
					1½φ-7½	¾φ-5	180									
			End	End	1½φ-6	¾φ-10	115									
					1½φ-6	¾φ-10	141									
			Interior	Interior	1½φ-6	¾φ-7	177	136								
5	63	0.417	Simple	End	1½φ-7½	¾φ-10	152	122								
					1½φ-7½	¾φ-10	159									
			End	End	1½φ-7½	¾φ-10	191	148	116							
					1½φ-7½	¾φ-10										
			Interior	Interior	1½φ-6½	¾φ-9	152	149								
5½	60	0.458	Simple	End	1½φ-6½	¾φ-6	193	149	151							
					1½φ-6½	¾φ-4½	245	191								
			End	End	1½φ-5½	¾φ-9	160	121								
					1½φ-5½	¾φ-8½	195	150	148							
			Interior	Interior	1½φ-5½	¾φ-6½	241	189								
6	75	0.500	Simple	End	1½φ-6½	¾φ-9½	208	161	132							
					1½φ-6½	¾φ-9½	218	169	161							
			End	End	1½φ-6½	¾φ-9½	260	204	161	128						
					1½φ-6½	¾φ-9										
			Interior	Interior	1½φ-5½	¾φ-8	203	199	157							
6½	81	0.542	Simple	End	1½φ-5½	¾φ-5	256	253	202	161						
					1½φ-5½	¾φ-4	213	164	127							
			End	End	1½φ-4½	¾φ-8	257	200	157	124						
					1½φ-4½	¾φ-7½	316	249	199	159	128					
			Interior	Interior	1½φ-4½	¾φ-6	274	215	169							
6	75	0.500	Simple	End	1½φ-5½	¾φ-8½	286	224	177	141						
					1½φ-5½	¾φ-8½		269	215	173	139					
			End	End	1½φ-5½	¾φ-8		203	159							
					1½φ-5	¾φ-7½	261	203	159							
			Interior	Interior	1½φ-5	¾φ-8½		257	204	162						
6	75	0.500	Simple	End	1½φ-5	¾φ-6½		213	259	209	170					
					1½φ-5	¾φ-6½		258	205	163	130					
			End	End	1½φ-4	¾φ-7	274	213	167	131						
					1½φ-4	¾φ-6½		256	207	168	136					
			Interior	Interior	1½φ-4	¾φ-5½		219	176							
6½	81	0.542	Simple	End	1½φ-5	¾φ-7½		275	219	176						
					1½φ-5	¾φ-7½		288	230	184	149					
			End	End	1½φ-5	¾φ-7½		276	276	224	183	149	122			
					1½φ-5	¾φ-7½		195	154							
			Interior	Interior	1½φ-5	¾φ-6½		248	199	161						
6½	81	0.542	Simple	End	1½φ-7½	¾φ-6½	317	248	195	154						
					1½φ-7½	¾φ-7½		311	248	199						
			End	End	1½φ-7½	¾φ-6		260	205	255	209	172	141			
					1½φ-6	¾φ-6½		250	201	163	129	130				
			Interior	Interior	1½φ-6	¾φ-6				201	162	169	138			
6½	81	0.542	Simple	End	1½φ-6	¾φ-6										
					1½φ-6	¾φ-6										
			End	End	1½φ-6	¾φ-6										
					1½φ-6	¾φ-6										
			Interior	Interior	1½φ-6	¾φ-6										







TABLE No. 22—TWO-WAY SOLID CONCRETE SLABS				RATIO OF SPANS 2.0 to 1.0								
SAFE SUPERIMPOSED LOADS IN LB. PER SQ. FT.				SPANS IN FEET								
Total Thick- ness of Slab  In.	Weight per Sq. Ft. Slab  Lb.	Volume of Concrete per Sq. Ft.  Cu. Ft.	Condition of Continuity		Reinforcing Steel Size and Spacing in Inches		20 x 10	22 x 11	24 x 12	26 x 13	28 x 14	30 x 15
			Short Way of Slab	Long Way of Slab	Short Way (Bottom Layer)	Long Way (Top Layer)						
4½	56	0.375	Simple End Interior	Interior End Interior	1½φ-7½ 1½φ-6 1½φ-6 1½φ-7½ 1½φ-7½	3⁄8φ-6 3⁄8φ-10 3⁄8φ-9 3⁄8φ-10 3⁄8φ-10	148 117 149 153 161	123				
5	63	0.417	Simple End Interior	Simple End Interior	1½φ-6½ 1½φ-6½ 1½φ-6½ 1½φ-5½ 1½φ-5½ 1½φ-5½ 1½φ-6½ 1½φ-6½ 1½φ-6½	3⁄8φ-9½ 3⁄8φ-7 3⁄8φ-5½ 3⁄8φ-9 3⁄8φ-9 3⁄8φ-8 3⁄8φ-9½ 3⁄8φ-9½ 3⁄8φ-9½	126 162 204 154 163 205 201 210 220	158 124 158 163 171	123 134			
5½	69	0.458	Simple End Interior	Simple End Interior	1½φ-5½ 1½φ-5½ 1½φ-5½ 1½φ-4½ 1½φ-4½ 1½φ-4½ 1½φ-5½ 1½φ-5½ 1½φ-5½	3⁄8φ-8½ 3⁄8φ-6 3⁄8φ-5 3⁄8φ-8 3⁄8φ-8 3⁄8φ-7 3⁄8φ-8½ 3⁄8φ-8½ 3⁄8φ-8½	170 216 269 206 217 270 267 277 289	129 167 210 158 167 211 209 217 227	166 129 166 171 180	131 143		
6	75	0.500	Simple End Interior	Simple End Interior	1½φ-5 1½φ-5 1½φ-5 1½φ-4 1½φ-4 1½φ-4 1½φ-5 1½φ-5 1½φ-5	3⁄8φ-7½ 3⁄8φ-5½ 3⁄8φ-4½ 3⁄8φ-7 3⁄8φ-6½ 3⁄8φ-7½ 3⁄8φ-7½ 3⁄8φ-7½ 3⁄8φ-7½	220 277 265 278	169 216 269 206 216 271 269 278 291	130 170 215 161 170 216 214 221 233	172 134 173 178 187	139	122
6½	81	0.542	Simple End Interior	Simple End Interior	1½φ-7½ 1½φ-7½ 1½φ-7½ 1½φ-6 1½φ-6 1½φ-6	3⁄8φ-7 1½φ-9½ 1½φ-7½ 3⁄8φ-6½ 3⁄8φ-6½ 3⁄8φ-6	268	207 263 251 263	161 209 198 209 263	125 166 211 157 166 212	170 171 132 171	138 139



7	88	0.583	Simple End Interior	Simple End Interior	<div> <div> <div>5/8φ-6 1/2</div> <div>1 1/2φ-8 1/2</div> <div>3/4φ-7</div> <div>3/8φ-6</div> <div>3/8φ-6</div> <div>3/8φ-5 1/2</div> <div>3/8φ-6 1/2</div> <div>3/8φ-6 1/2</div> <div>3/8φ-6 1/2</div> </div> <div> <div>124</div> <div>159</div> <div>202</div> <div>257</div> <div>308</div> </div> </div>	<div> <div>124</div> <div>159</div> <div>202</div> <div>257</div> <div>308</div> </div> <div> <div>133</div> <div>173</div> </div>
7 1/2	94	0.625	Simple End Interior	Simple End Interior	<div> <div>5/8φ-6</div> <div>5/8φ-6</div> <div>5/8φ-6</div> <div>5/8φ-5</div> <div>5/8φ-5</div> <div>5/8φ-5</div> <div>5/8φ-6</div> <div>5/8φ-6</div> <div>5/8φ-6</div> </div> <div> <div>155</div> <div>195</div> <div>246</div> </div>	<div> <div>123</div> <div>165</div> <div>212</div> <div>156</div> <div>166</div> <div>214</div> <div>212</div> <div>220</div> <div>232</div> </div> <div> <div>155</div> <div>204</div> <div>258</div> <div>193</div> <div>204</div> <div>260</div> <div>257</div> <div>267</div> <div>280</div> </div>
8	100	0.667	Simple End Interior	Simple End Interior	<div> <div>5/8φ-5 1/2</div> <div>5/8φ-5 1/2</div> <div>5/8φ-5 1/2</div> <div>5/8φ-4 1/2</div> <div>5/8φ-4 1/2</div> <div>5/8φ-4 1/2</div> <div>5/8φ-5 1/2</div> <div>5/8φ-5 1/2</div> <div>5/8φ-5 1/2</div> </div> <div> <div>190</div> <div>226</div> <div>295</div> </div>	<div> <div>153</div> <div>202</div> <div>257</div> <div>191</div> <div>202</div> <div>258</div> <div>255</div> <div>266</div> <div>279</div> </div> <div> <div>190</div> <div>246</div> <div>234</div> <div>247</div> </div>
8 1/2	106	0.708	Simple End Interior	Simple End Interior	<div> <div>3/4φ-8</div> <div>3/4φ-8</div> <div>3/4φ-8</div> <div>3/4φ-6 1/2</div> <div>3/4φ-6 1/2</div> <div>3/4φ-6 1/2</div> <div>3/4φ-8 1/2</div> <div>3/4φ-8</div> <div>3/4φ-8</div> </div> <div> <div>274</div> </div>	<div> <div>180</div> <div>235</div> <div>297</div> <div>223</div> <div>235</div> <div>299</div> <div>295</div> <div>307</div> <div>323</div> </div> <div> <div>222</div> <div>285</div> <div>271</div> <div>286</div> </div>
9	113	0.750	Simple End Interior	Simple End Interior	<div> <div>3/4φ-7 1/2</div> <div>3/4φ-7 1/2</div> <div>3/4φ-7 1/2</div> <div>3/4φ-6</div> <div>3/4φ-6</div> <div>3/4φ-6</div> <div>3/4φ-7 1/2</div> <div>3/4φ-7 1/2</div> <div>3/4φ-7 1/2</div> </div> <div> <div>261</div> </div>	<div> <div>212</div> <div>276</div> <div>346</div> <div>262</div> <div>276</div> <div>348</div> <div>344</div> <div>358</div> <div>375</div> </div> <div> <div>261</div> </div>

1-r For Shear	
For Long Beam	For Short Beam
0.890	0.110
0.841	0.159
0.778	0.222
0.924	0.076
0.890	0.110
0.843	0.157
0.948	0.052
0.924	0.076
0.890	0.110

1-er For Bending	
For Long Beam	For Short Beam
0.936	0.111
0.905	0.255
0.865	0.370
0.957	0.077
0.936	0.111
0.907	0.250
0.971	0.052
0.957	0.078
0.936	0.111



## SECTION IV—SPECIAL DETAILS OF CONSTRUCTION

### *Openings in Slabs*

Openings of various sizes and shapes will be encountered in any floor slab, due to stairs, elevator shafts, pipe shafts, and ventilating ducts. The method of providing for these depends largely on the ingenuity of the designer. Effort should always be made to preserve the continuity of the floor as far as possible and to eliminate framing beams or girders if it can be done without sacrificing stiffness and safety as a whole.

Small openings usually require merely a rearrangement of the reinforcing steel and the introduction of a few additional bars. A right and a wrong method are illustrated in Fig. 18.

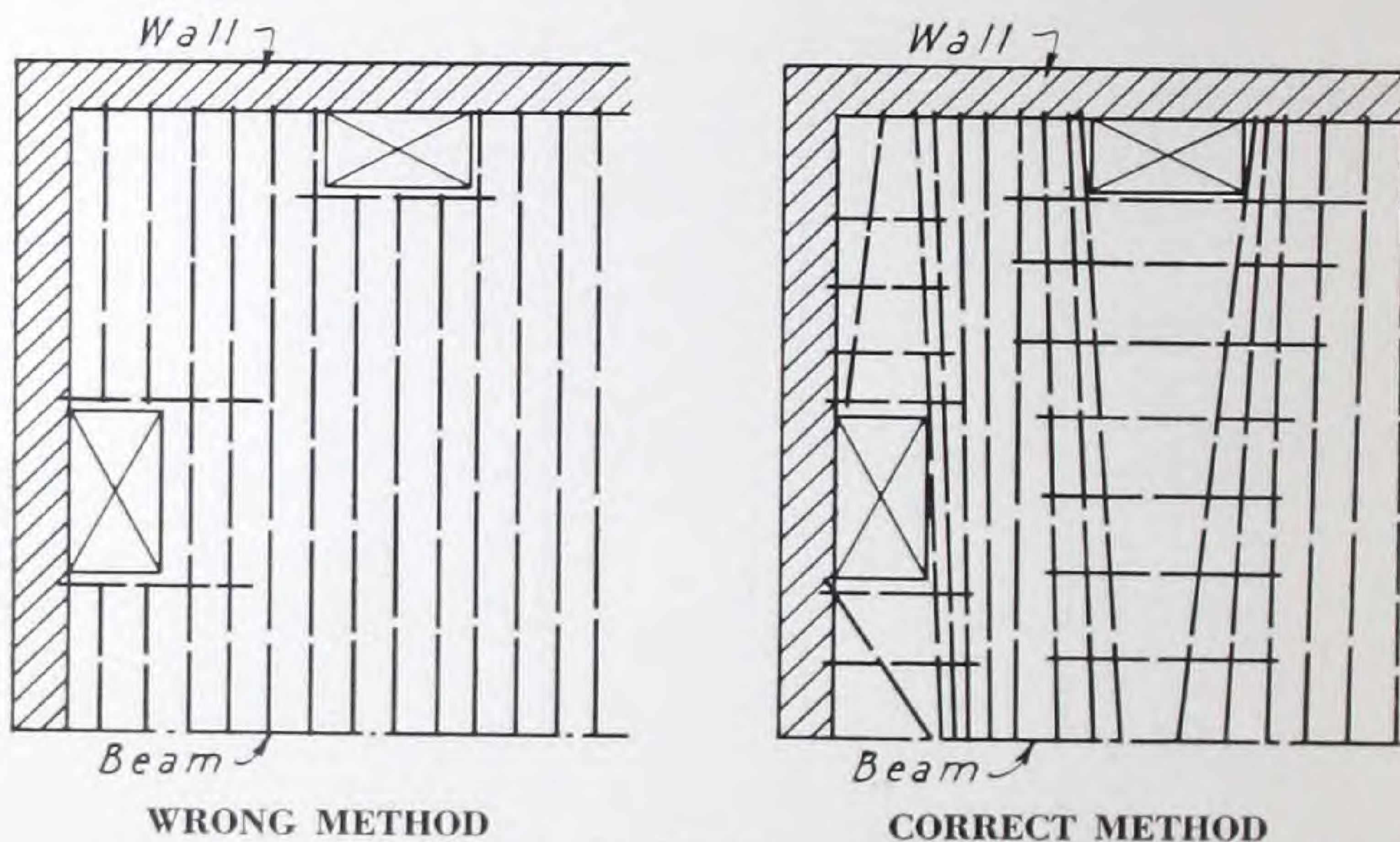


Fig. 18

Larger openings may sometimes be framed in one-way ribbed slabs by introducing header joists at right angles to the main joists and framing into wider or double joists along the sides of the opening.

Very large openings may be more economically framed by marginal beams, which in turn frame into the main supporting beams or girders.

### *Stairs and Stair Platforms*

Reinforced concrete stairs usually consist of an inclined slab with the steps formed on its upper surface. These may be quite simply designed by reference to the following table.



TABLE No. 23—TABLE OF CONCRETE STAIR SLABS

$f_s = 20,000$  p.s.i.  
 $f_c = 800$  p.s.i.  
 $n = 15$   
 $L.L = 100$  p.s.f.

1" Clear Protective  
Covering  
Plastered Soffit

Horizontal Span of Stairs in Feet	Total Thickness of Slab in Inches	Reinforcing Steel
4	3	$\frac{3}{8}" \phi - 7\frac{1}{2}"$ o.c.
5	$3\frac{1}{2}$	$\frac{3}{8}" \phi - 6\frac{1}{2}"$ o.c.
6	4	$\frac{3}{8}" \phi - 5\frac{1}{2}"$ o.c.
7	$4\frac{1}{2}$	$\frac{3}{8}" \phi - 4\frac{1}{2}"$ o.c.
8	5	$\frac{1}{2}" \phi - 7\frac{1}{2}"$ o.c.
9	$5\frac{1}{2}$	$\frac{1}{2}" \phi - 6\frac{1}{2}"$ o.c.
10	6	$\frac{1}{2}" \phi - 5\frac{1}{2}"$ o.c.
11	$6\frac{1}{2}$	$\frac{1}{2}" \phi - 5"$ o.c.
12	7	$\frac{5}{8}" \phi - 7"$ o.c.
13	$7\frac{1}{2}$	$\frac{5}{8}" \phi - 6\frac{1}{2}"$ o.c.
14	8	$\frac{5}{8}" \phi - 6"$ o.c.
15	9	$\frac{5}{8}" \phi - 5\frac{1}{2}"$ o.c.
16	$9\frac{1}{2}$	$\frac{3}{4}" \phi - 7"$ o.c.
17	10	$\frac{3}{4}" \phi - 6\frac{1}{2}"$ o.c.
18	$10\frac{1}{2}$	$\frac{3}{4}" \phi - 6"$ o.c.
19	$11\frac{1}{2}$	$\frac{3}{4}" \phi - 5\frac{1}{2}"$ o.c.
20	12	$\frac{3}{4}" \phi - 5\frac{1}{2}"$ o.c.

In stair flights which incorporate small platforms, care should be taken that the reinforcing steel is properly placed at the junction with the steps, particularly where the platform is at the top of the flight. Fig. 19 clearly shows this detail.

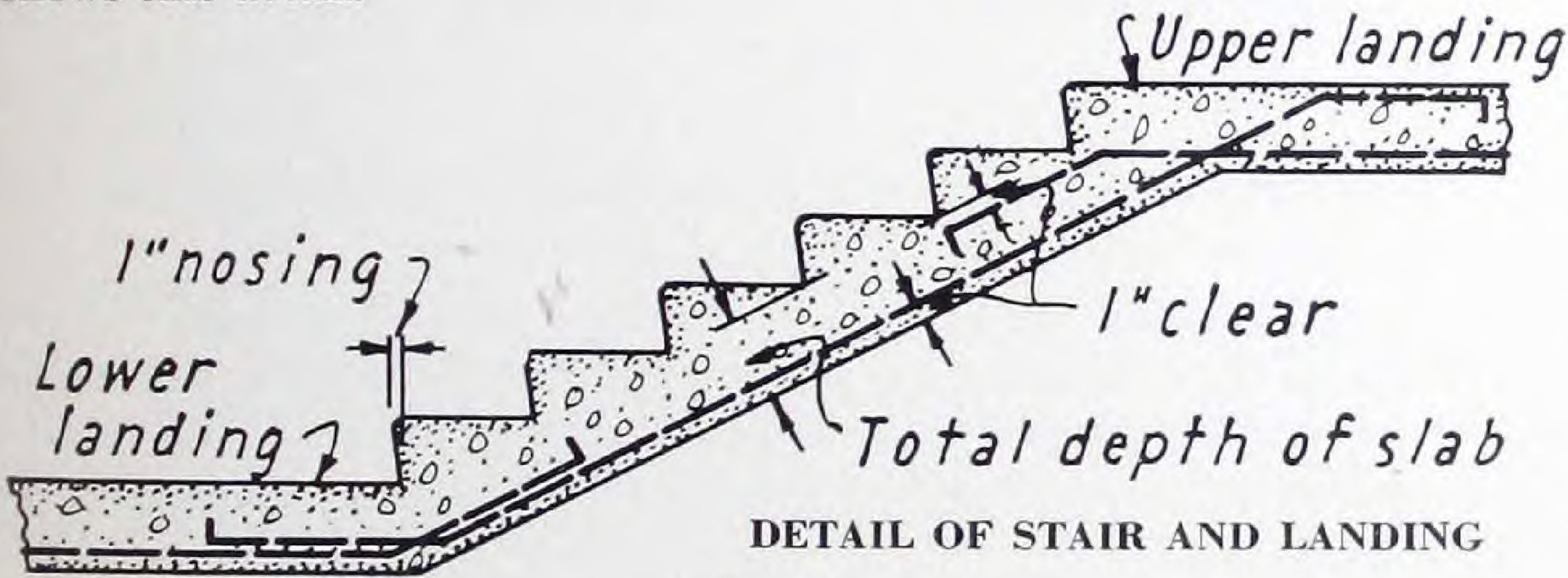


Fig. 19

Methods of Anchoring Reinforcement  
in Discontinuous Slabs

There are several ways of anchoring bars at the edges of discontinuous slabs or beams. One is to extend the bar into the concrete of the support for such a distance that the stresses in the bar are gradually transferred by bond to the concrete of the support.

In cases where the support is not large enough to permit a bar to develop its bond, a hook should be provided at the end. Fig. 20 illustrates the right



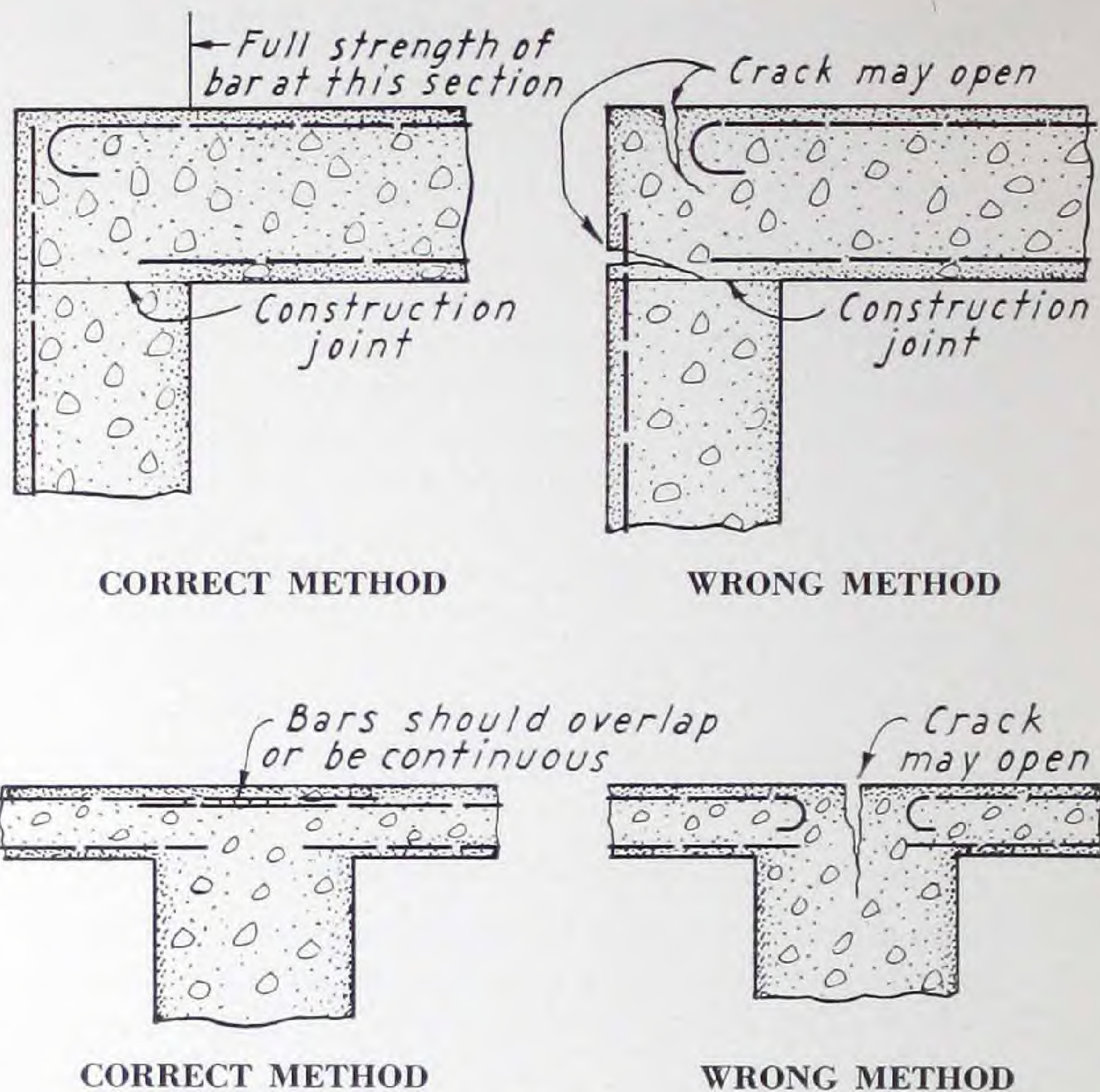


Fig. 20

and wrong method of providing such anchorage. The effectiveness of a hook depends partly upon the strength of the concrete.

The ideal anchorage is obtained by a combination of straight embedment equal to 15 diameters of the bar and a semi-circular hook, the inside diameter of which is at least 4 times the diameter of the bar. With such a detail, the elastic limit of the steel can be reached without causing excessive stresses in the concrete.

### **Construction Joints**

Construction joints will be necessary when the placing of concrete cannot be done continuously. They should be near the center of spans of beams and slabs, where the shear is a minimum. Bulkheads at right angles to the reinforcement prevent the last concrete placed from "feathering" out and provide a good surface for the further deposit of concrete. In ribbed slabs with metal pans, the bulkheads should be perpendicular to the joists. If they are set in the opposite direction, cracks sometimes develop at the joints.



**TABLE No. 24 — RECOMMENDED LIVE LOADS**

	Lb. Per Sq. Ft.
Apartments . . . . .	40
Armories . . . . .	150
Auditoriums—fixed seats . . . . .	50
Auditoriums—movable seats . . . . .	100
Balconies and galleries—fixed seats . . . . .	50
Balconies and galleries—movable seats . . . . .	100
Dance halls . . . . .	120
Dwellings . . . . .	40
Exterior balconies . . . . .	100
Fire escapes . . . . .	100
Garages—all types of vehicles . . . . .	100
Garages—passenger cars only . . . . .	80
Gymnasiums . . . . .	100
Hospitals—wards and rooms . . . . .	40
Hospitals—corridors and public rooms . . . . .	100
Hotels—guest rooms and corridors . . . . .	40
Hotels—public rooms and public corridors . . . . .	100
Libraries—reading rooms . . . . .	50
Libraries—corridors . . . . .	100
Loft buildings . . . . .	100
Manufacturing—light . . . . .	75
Manufacturing—heavy . . . . .	125
Offices . . . . .	50
Printing plants—composing and linotype rooms . . . . .	100
Public rooms . . . . .	100
Reviewing stands, bleachers, grandstands, etc. . . . .	100
Roof loads—rise less than 1 to 4 . . . . .	30
Schools—class rooms . . . . .	50
Schools—corridors . . . . .	100
Sidewalks—800 lb. concentrated or . . . . .	250
Stairways . . . . .	100
Storage—light . . . . .	100
Storage—heavy, not less than . . . . .	250
Stores—retail (light merchandise) . . . . .	75
Stores—wholesale (light merchandise) . . . . .	100
Theatres—stage floor . . . . .	100



TABLE No. 25—WEIGHTS OF BUILDING MATERIALS

	Lb. Per Sq. Ft.
Asphalt (2" thick)	25
Brickwork, common (per inch of thickness)	10
Cement floor finish (1" thick)	12
Cinders (1" thick)	3½
Composition roof	10
Concrete, cinder (1" thick)	7
Concrete, stone or gravel (1" thick)	12½
Continuous steel sash, glazed	8-10
Creosoted wood blocks (4" thick)	20
Earth (12" thick)	100
Gypsum (1" thick)	3
Hollow clay tile (3" thick)	15
Hollow clay tile (4" thick)	16
Hollow clay tile (6" thick)	22
Hollow clay tile (8" thick)	30
Hollow clay tile (10" thick)	36
Hollow clay tile (12" thick)	40
Lightweight Concrete Tile—5" x 4" x 12"	14
Lightweight Concrete Tile—5" x 6" x 12"	22
Lightweight Concrete Tile—5" x 8" x 12"	28
Limestone masonry (12" thick)	160
Marble masonry (12" thick)	170
Partitions, 2 x 4 stud, plastered both sides	16
Partitions, solid plaster (2" thick)	20
Plaster, on tile or concrete	5
Plaster, on metal lath	10
Polished plate glass	3½
Pressed steel ceilings	2
Quarry tile (1" thick)	12
Sand (1" thick)	9
Sheathing, Y.P. (1" thick)	4
Sidewalk lights	40
Skylights, metal, with wire glass	6
Slate roof	6
Steel sash, glazed with S.S. glass	5
Terrazzo floor (2" thick)	25







